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WATER-WORKS

FOR

SMALL CITIES AND TOWNS

BY

JOHN GOODELL

McGRAW-HILL BOOK COMPANY

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Introductory Note.

The following notes on small water-works have been compiled to meet the desire for such information which is shown by letters addressed to "The Engineering Record." The book contains no new theories and no references to methods of construction or design which have not proved satisfactory in actual use. As an offset to this lack of originality there will be found in the following pages considerable information never before collected in a single volume, and troublesome to obtain elsewhere. The editing of this material has been done with the idea of making the result of service to water-works trustees as well as superintendents and engineers; this will account for the attention paid to some details which technically educated officials may consider very elementary.

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WATER-WORKS FOR SMALL CITIES AND TOWNS.

CHAPTER I.—SURFACE WATER.

By surface water is meant the water discharged from the surface of a catchment area, as opposed to that collected from wells and galleries. Such surface supplies depend upon the rainfall for their existence, and upon the natural features of the watershed for their character. It is generally an easy matter to secure statistics of the rainfall in the neighborhood of most places large enough to have water-works, and from such statistics and an inspection of the catchment area the probable amount of water usually available may be determined. It is sometimes possible, also to secure maps of the watershed, which are often of much value as indicating the sources from which a supply may be obtained; the maps of the national and state geological surveys are frequently used for this purpose.

While it is the exception rather than the rule that a survey of a watershed for a small system has to be made with accuracy and be mapped subsequently, yet cases may arise when this is necessary, and consequently a few hints are given here as to the methods now regarded as best adapted for such work. It is pretty generally believed by engineers who have done most work of this sort that the quickest and cheapest method is by means of a transit having stadia hairs in the telescope, the notes being plotted in the field on a sketchboard. The accuracy of the stadia was definitely settled during the survey of the Mexican boundary a few years ago, and if the precautions are followed which are recommended in a paper on this survey by J. L. Van Ornum, Assoc. M. Am. Soc. C. E. (see the "Transactions" of the American Society of Civil Engineers, Vol. xxxiv., p. 259), the topography of a watershed may be taken with much dispatch and sufficient accuracy for all

purposes. It is unnecessary here to give instructions as to the method of making the stadia survey, but a few notes on the use of a sketchboard may be appropriate, as the published information on the subject is meager and not readily secured.

The sketchboard is merely a portable drawing-board on which the transit and stadia notes are plotted in the field as fast as practicable. These notes furnish the location and elevation of all the leading features of the tract surveyed, and it is an easy matter for a draughtsman to sketch in the intervening contour lines by eye. One of the best sketchboards known to the writer was designed by A. R. Paddock, and is described in "The Engineering Record" for October 31, 1891, and February 6, 1892. The top is a wooden frame, into which fit four small wooden squares. The top of each square is covered with drawing-paper, which is turned over the edges and fastened to the lower surface. When the four squares covered in this way are placed in the frame a uniform paper surface of about 310 square inches is furnished for sketching. Whenever the plotting approaches one edge of the board, as the east, the two eastern sections are transferred to the west edge and two fresh ones put in their place. The field notes were plotted on these tables in two ways. Some were laid off by using the magnetic bearings and marking off the points with a card protractor having a scale on one edge, while others were fixed by using a simple alidade consisting of a metallic triangular scale with a pair of sighting wires attached. The cost of a table of this sort was about $11.

On the surveys made in 1884, under the direction of Rudolph Hering, M. Am. Soc. C. E., in connection with his studies for a new water supply for Philadelphia, the advantages of the sketching-table and transit method of surveying were found to be as follows:

"First, the immediate detection and correction of errors in angles or measures while plotting, thereby saving delays and inaccuracies in the office work; secondly, the drawing and sketching of the contours in greater conformity with the shape of the ground while viewing it, and saving the measuring of many points that would be necessary in the case of office plotting; thirdly, the possibility of putting in minor points by sight intersections, as on a plane table; and fourthly, of having the map so far completed in the field that it requires little more than an adjustment and transfer by means of blackened paper to be in its proper form

and place upon the section maps. The results have so far been very satisfactory, the plotting shows a high degree of accuracy, and the gain in time has been considerable. In the field the plotting parties could, on similar territory, of course not cover as much ground per week as the others, their gain being in office work. In fair country it was found possible readily to survey and plot in the field, to a scale of 400 feet to 1 inch, 3 square miles per week, the party consisting of the engineer and one rodman." (Report by Rudolph Hering.)

On this survey the field table was 15 inches square, and had a universal joint and tangent screw. It rested on a very light tripod. The sheets of paper were ruled to quarter-inch squares, each square representing a 100-foot square of land. Main lines were run with the transit by magnetic courses and stadia in circuits of 1 to 3 miles, and these were immediately plotted to scale on the table standing beside the transit. The levels were taken by the transit, by spirit leveling and vertical angles, from bench marks established by a level party, and were plotted at once on the paper, so that the contours could be sketched in. Buildings, roads, and streams were located by transit angles and then plotted, or were sighted from the table, after leveling and orienting it. Colored pencils were used to mark in certain features; blue for watercourses, green for the outlines of woods, and yellow for roads.

Of course it is not always convenient to use this method of surveying, and when a sketching-table is not available a modification of the following method may be found advantageous. This method was also used under Mr. Hering's direction. The instruments employed were a transit, stadia rod, slope level, prismatic compass, and barometer. Main or base lines, with magnetic courses and stadia distances, were run over the territory, so as to form approximate quadrilaterals of about one-half to three-fourths square mile each. They were bounded by definite lines, as roads or streams, tied at the four corners. The position and elevation of high and low points was taken by magnetic and vertical angles, and stadia distances directly from the main line, if possible, otherwise a spur line was run to where they could be seen. In thickly wooded regions the topography was taken with the prismatic compass, slope level, and barometer. Buildings were located either by at least two magnetic angles from the base

line, or one angle and stadia measurement. Roads, streams, and such features were fixed in the latter way. Levels were carried in the manner mentioned in connection with the sketch-table method, and were generally found to check within a foot in a circuit of 2 or 3 miles. The vertical angles were reduced by natural sines and cosines on the main lines, and by co-ordinate paper protractors as well on the laterals. All notes and sketches were entered in a transit-book.

The amount of water that may be obtained from a catchment area is very variable and can only be estimated roughly. A small area, under about 2 square miles in extent, may be practically dry at times. Statistics compiled under the direction of the Boston Water Board show that from 40 to 50 per cent. of the rainfall has been collected, on an average, for many years on the watersheds under its control. These statistics are probably the most complete in this country, and on this account they are largely used in estimating the flow from catchment basins. The characteristics of the three watersheds controlled by the Boston Water Board are described as follows by Desmond FitzGerald, M. Am. Soc. C. E.:

"The Sudbury River watershed has an area of 75.199 square miles; the Mystic, 26.9 square miles; and the Cochituate, 18.87 square miles. The Sudbury is hilly, with steep slopes. There are, however, some large swamps within its borders. The Cochituate, although adjoining the Sudbury, is entirely dissimilar. The slopes are flat and sandy. The surface is mostly modified drift, while the Sudbury is largely composed of unmodified drift. The Mystic watershed lies to the north of Boston, and about 30 miles distant from the other two sources, which are to the west of the city. Its surface is steeper than the Cochituate and not as steep as the Sudbury."

The average monthly yield per square mile of these watersheds for a period of 13 years was as follows:

Month.	Gallons.	Month.	Gallons.
January	37,387,000	July	7,491,000
February	55,056,000	August	11,399,000
March	71,226,000	September	10,242,000
April	49,107,000	October	16,797,000
May	30,406,000	November	24,787,000
June	14,975,000	December	34,128,000

These figures are interesting and valuable in many ways as illustrating the monthly variations in stream flow under conditions

that will probably be found to agree approximately with those in many other parts of the United States. The average rainfall at Boston for the last 75 years has been as follows: January, 3.98 inches; February, 3.78; March, 4.36; April, 4.06; May, 3.79; June, 3.27; July, 3.71; August, 4.39; September, 3.55; October, 3.84; November, 4.31; December, 3.96; total, 47 inches.

In the design of water-works depending upon surface supplies averages are not of as much importance as minimums, and on this account the Sudbury River statistics are of great value. They extend back many years and contain records of two periods of remarkable drought. Engineers want to know the minimum quantities of water to expect from a watershed, and the following table is therefore given showing the daily flow from the Sudbury basin for different periods, the figures giving the daily gallons per square mile in the driest period of the given duration:

Period.	Gallons.	Period.	Gallons.	Period.	Gallons.
1 month....	44,000	7 months..	147,000	2 years...	687,000
2 months...	64,000	8 months..	181,000	3 years...	764,000
3 months...	95,000	9 months..	219,000	4 years...	735,000
4 months...	118,000	10 months..	312,000	5 years...	769,000
5 months...	131,000	11 months..	409,000	6 years...	803,000
6 months...	143,000	1 year.....	497,000	7 years...	839,000

The average flow from the Sudbury River watershed per square mile during the driest periods of five years or less has been, so far back as records have been kept, very much less than from the Croton watershed in New York, while the average flow for the whole 19 years of observation on both basins has been nearly the same.

It is evident from the above table that if means can be provided for storing the surplus water from a catchment area during the periods when the supply exceeds the consumption, it will be possible to satisfy daily drafts of many times the yield of the watershed during the driest periods that are liable to occur. In the "Transactions" of the American Society of Civil Engineers, Vol. xxvii., p. 265, Mr. FitzGerald gave a diagram of the storage capacity required to sustain drafts of 100,000 to 900,000 gallons daily from 1 square mile of watershed containing various percentages of water surface. F. P. Stearns, M. Am. Soc. C. E., has prepared Table No. 1 for the same purpose, arranged in a somewhat different manner and taking evaporation into account.

The nature of the investigations upon which this table was

based was explained in "The Engineering Record" of February 24, 1894, and engineers who desire to study the matter farther are referred to that article and to the paper by W. Rippl entitled "The Capacity of Storage Reservoirs for Water Supply," which was published in the "Proceedings" of the Institution of Civil Engineers, Vol. lxxi., p. 270.

Table No. 1.—Showing the Amount of Storage Required to Make Available Different Daily Volumes of Water per Square Mile of Watershed (Estimating Land Surfaces Only), Corrected for the Effect of Evaporation and Rainfall on Varying Percentages of Water Surface, not Included in Estimating the Area of the Watershed.

Daily Volume in Gallons per Square Mile of Land Surface.	Storage Required in Gallons per Square Mile of Land Surface to Prevent a Deficiency in the Season of Greatest Drought When the Daily Consumption is as Indicated in the First Column, with the Following Percentages of Water Surfaces.				
	0	3	6	10	25
100,000	556,000	3,000,000	8,800,000	....	....
150,000	3,400,000	7,100,000	13,400,000	....	....
200,000	9,400,000	11,700,000	18,000,000	....	....
250,000	19,000,000	22,200,000	25,400,000	....	....
300,000	29,800,000	33,000,000	36,100,000	....	....
400,000	52,000,000	54,400,000	57,500,000	....	....
500,000	76,500,000	77,300,000	80,300,000	....	....
600,000	102,000,000	104,600,000	107,100,000	112,800,000	....
700,000	144,400,000	153,000,000	161,600,000	170,700,000	215,900,000
800,000	202,300,000	210,900,000	219,500,000	228,600,000	273,800,000
900,000	346,200,000	349,200,000	352,200,000	353,900,000	381,600,000
1,000,000	514,600,000	516,700,000	519,700,000	523,600,000	532,200,000

The manner in which the table is to be used can be most easily indicated by quoting from Mr. Stearns's explanation.

"Let us assume that it is desired to know the yield of a pond having an area of 0.15 square mile and an available storage capacity of 225,000,000 gallons, which has draining into it 1.5 square miles of land surface. The amount of storage in this case would be equivalent to 150,000,000 gallons per square mile of land surface, and the water surface would equal 10 per cent. of the land surface. By looking in the column of the table headed 10 per cent., it will be seen that a storage of 150,000,000 gallons per square mile corresponds to a daily volume of between 600,000 to 700,000 gallons per square mile, or more exactly, by proportion, to 660,000 gallons, equal to 990,000 gallons daily for the whole watershed. The results obtained by this method will in some cases be practically correct. In other cases

it will be necessary to take account of local conditions, prominent among which may be leakage past a dam, or filtration through the ground to lower levels; and the application of judgment will often be necessary to determine whether the watershed under consideration will yield the same or a greater or a less amount per square mile than that of the Sudbury River."

The more frequent problem, however, is to determine upon the amount of storage required to enable a definite quantity of water to be drawn from a given watershed. For instance, suppose 1,000,000 gallons a day are wanted from a watershed having a land area of 1.5 square miles. There is hardly a catchment area large enough to be considered as a collecting ground for water-works that has not some water surface, and Mr. FitzGerald says that in the use of such methods of estimating as are here considered, it is of doubtful utility to consider anything under 2 per cent. In the present case it will be assumed that the only water on the catchment area is a brook of undetermined but small superficial extent; accordingly 2 per cent. will be assumed as the water area. The amount of water required per square mile of land area will be a little under 670,000 gallons. By interpolating from the table it will be found that the storage needed under these conditions will be about 136,300,000 gallons per square mile of land surface, or 235,000,000 gallons all told. Such a storage volume, however, would require a water surface of, say, about 0.13 square mile. This, however, is about 8½ per cent, of the total land area, and indicates that another computation will be needed.

In view of the results of the first calculation it will be assumed that the land area is reduced to 1.4 square miles by reason of the construction of the reservoir, and that the water area is 10 per cent. of the land area. In this case, the draft per square mile of land surface will be a little over 700,000 gallons a day, and will require a storage capacity of 171,000,000 gallons per square mile, or 240,000,000 gallons all told. Hence such a reservoir under the conditions assumed may be considered adequate to supply the quantity of water desired. But such a reservoir might prove very undesirable from another point of view, even if a location for it can be found.

It has been pointed out repeatedly that any attempt to store more than 700,000 gallons per square mile of watershed in artificial reservoirs, under the conditions obtaining in the Sudbury

River basin, will be very liable to end in failure from the fact that during several consecutive months the water level in the reservoir may be so low as to permit a growth of weeds on the exposed shores, which will cause a marked deterioration in the quality of the water.

Mr. Stearns states that taking everything into account it may be said the greatest amount which can be made practically available from a square mile of watershed does not exceed 900,000 gallons per day, and the cases are very rare in which more than 600,000 gallons per square mile per day can be made available when it is necessary to store the water in artificial reservoirs. Mr. FitzGerald states that it is impracticable to secure more than about 750,000 gallons daily per square mile of watershed containing 10 per cent. of water surface. The reason for this limitation is that the levels in the reservoir should not be made to fluctuate too much and that the reservoir should not be drawn below the high-water mark for too long a time.

The above method of estimating the amount of water which may be made available from a watershed is very conservative. Under the conditions of rainfall and topography upon which it is based, it will be too conservative for most years. But a water famine is a serious affair, and it is well to provide against it. The modifications of the method to suit localities where the conditions of rainfall differ from those mentioned are matters of judgment for which no general rules can be given. It must not be forgotten, however, that because the discharge of small catchment areas is liable to so much greater variations than that of larger basins, it is highly important to avoid overestimating their yield in periods of extreme drought. Speaking generally, it may be said that unless a watershed is very mountainous, very flat, or very sandy, or unless the rainfall upon it differs considerably during a term of several years from the averages given in the preceding discussion, the determination of the available supply by the foregoing method will give results as nearly accurate as such estimates can be made. Moreover, the determination thus made will be free from the very frequent error of exaggerating the amount of water that may be safely counted upon.

In some cases there may be a brook flowing from the watershed, and it is desirable to ascertain roughly how much water is passing in it. In such a case the cross-section of the stream should be

measured as carefully as possible at the lower end of the straightest and most uniform part. Then a distance of 100 feet should be measured back from the place where the cross-section was taken, and the number of minutes it takes several small pieces of wood to pass this 100 feet should be observed. In this way the surface velocity of the brook may be determined. Eight-tenths of this velocity, expressed in feet per minute, multiplied by the cross-section of the brook in square feet, will give approximately the discharge of the brook in cubic feet a minute. This method should not be used unless the 100-foot section of the brook is fairly straight and differs but little in its sectional area throughout the distance.

Where greater accuracy is desired weirs must be employed. A

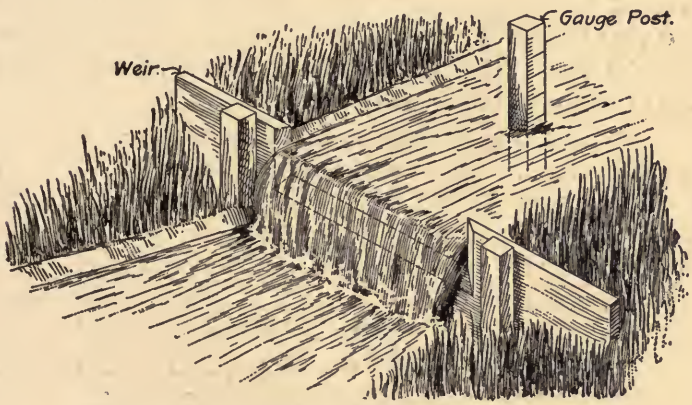


FIGURE 1.—ARRANGEMENT FOR GAUGING BROOKS.

rough weir can be made as shown in Figure 1 without much difficulty. The planks should be firmly bedded in the bottom and sides of the brook, and the three edges of the rectangular notch should be beveled off to an angle. The distance from the sides and bottom of the notch to the banks and the bed of the brook should be not less than three times the depth of the water above the lower edge of the notch. The bottom edge of the notch should be perfectly level. Care must be taken that water does not have an opportunity of leaking under or around the weir. Six feet or more above the weir a stake should be driven firmly in the bed of the brook. When this is done, which should be before the planks are put in place, the weir should be built up, and the ele-

vation marked very carefully on the stake at which water begins to flow over the notch. Then a scale of inches should be marked on the stake with this point as a zero. If the work is well done this scale will enable the depth of water above the notch to be determined quite accurately. In order to determine the flow over the weir, take the reading in feet and decimals of a foot on the stake and then obtain from Table No. 2 the discharge in cubic feet per second over a weir 1 foot long for that depth. Multiply this discharge by the length of the notch in feet and the result will be the discharge of the brook in cubic feet per second. Care should be taken to make the notch of such a size that the flow through it will not exceed 6 or 8 inches a second, if possible. Notches 2 feet long are best with depths of water of 0.3 to 0.7 foot, 3 feet long with depths of 0.3 to 1 foot, and weirs 5 feet long with depths as high as 1.7 feet. The values in the table are to be understood as approximations, suitable, however, for the rough nature of such measurements as are here descibed:

Table No. 2.—Discharge of Weirs 1 Foot Long.

Depth, ft.....	0.4	0.45	0.5	0.55	0.6	0.65
Discharge	0.836	0.998	1.167	1.345	1.531	1.724
Depth, ft.....	0.7	0.75	0.8	0.85	0.9	0.95
Discharge	1.925	2.133	2.347	2.568	2.795	3.028
Depth, ft.....	1.0	1.05	1.1	1.15	1.2	1.25
Discharge	3.267	3.511	3.761	4.016	4.277	4.542
Depth, ft.....	1.3	1.35	1.4	1.45	1.5	1.55
Discharge	4.812	5.087	5.367	5.051	5.940	6.233
Depth, ft.....	1.6	1.65	1.7	1.75	1.8	1.85
Discharge	6.530	6.832	7.137	7.447	7.760	8.077
Depth, ft.....	1.9	1.95	2.0	..	..	..
Discharge	8.398	8.723	9.051	..	..	..

The quality of water from a watershed depends upon the population of the area, the number of swamps in it, and the nature of the rock formation over which the water passes. If limestone is common, the water is liable to be too hard, if there are swamps on the catchment area, the color is apt to be dark, and if the population on the watershed is more than about 300 per square mile, the stored water often proves troublesome from bad tastes and odors. "Shallow storage reservoirs, from which the soil and vegetable matter have not been removed, generally give trouble, and the large and deep reservoirs of the same character are by no means exempt."

The suitability of water for a town and municipal supply depends upon its freedom from sewage contamination, hardness,

color, odor, and taste. The direct sewage contamination of a surface supply rarely needs to be determined by analysis, as the inspection of the watershed will indicate any sources of pollution. Where water is drawn from lakes and streams the aid of the chemist and biologist must be sought before judgment can be passed safely on any water. These analysts have generally printed directions as to the methods by which samples are to be collected, so it is hardly necessary to give such instructions here. In sending the samples full information should be furnished regarding the nature of the surroundings of the places where they were obtained, as the analyst is thereby enabled to come to more definite conclusions regarding the availability of the source for the purposes proposed.

Few water commissioners and superintendents are not confronted sooner or later with apparently formidable tables of chemical analyses, so it may be well to point out what such analyses mean. The source of the most serious pollution of water is organic matter. This may be present as living organisms and the product of organic life, or the matter may be present in various stages of decomposition. It is customary to classify the condition of the organic matter by means of the condition of the nitrogenous organic matter. In this way the albuminoid ammonia is taken as an indication of the amount of undecomposed organic matter. When decomposition has begun, its extent is indicated by the presence of so-called free ammonia. Further changes result in altering the free ammonia to nitrites, which finally become nitrates, the last stage in the process of alteration by which organic matter is converted into a form suited for assimilation into new organic matter. It is imprudent to state that because a water contains unusually large amounts of any of these compounds of nitrogen that it is necessarily polluted. The signification of each compound may be stated briefly as follows, it being understood that only surface waters are now under consideration:

Albuminoid ammonia was formerly considered as an indication of the presence of an equivalent amount of organic matter liable to decay, but within recent years it has been found that this it not necessarily so. The lesson to be learned from this compound is indicated most clearly by successive analyses of a water, for if the albuminoid ammonia remains unchanged for months without developing free ammonia, a comparatively large amount may be

harmless. This is especially the case with brown coloring matter which water dissolves from grasses, leaves and roots, according to Dr. T. M. Drown, who instances the very dark water of Acushnet River, the source of New Bedford's supply, as a good water containing enough albuminoid ammonia to be classed as polluted by most European standards.

Free ammonia, so-called, is a characteristic ingredient of sewage, but "the conditions which influence its development and accumulation in natural waters are so various that one must be extremely cautious in deciding what is the signification of its presence in individual cases." If an analysis shows a large amount of free ammonia in a water from a catchment area having dwellings upon it, further investigations should be made into the causes of its presence.

Nitrites are compounds of much interest, as their amount is generally found to vary less with the seasons than the other organic derivatives, and they are therefore a better index of sewage pollution. "High free ammonia and high nitrites together are characteristic of recent pollution, and when they are uniformly high in a surface water they point to continuous pollution."

Nitrates indicate the complete change of organic to inorganic matter, and their importance can only be settled satisfactorily when the source from which they were derived is known. The organic matter that is discharged into a water is rarely dangerous if it is given time to change to nitrates, but the disease germs that may have been discharged at the same time may be still a source of danger when the chemical changes are over. Chemical analysis, by indicating the amount of albuminoid and free ammonia, nitrites and nitrates, points out the probability of such germs being in the water and the time that has elapsed since they were discharged into it. The time is probably least when the albuminoid ammonia is high, and greatest when the nitrates are high in the analysis.

Chlorine is a valuable indication of sewage contamination. The amount to be found in unpolluted water varies widely; in Massachusetts it decreases as the distance from the seashore increases, and it is highly desirable to know what is the normal amount in unpolluted water in a given region before deciding upon the signification of the amount shown by analyses of samples of water of unknown character from the same locality. The chlorine in

the reservoirs of the Boston water system is found to vary directly with the population upon their respective watersheds. High free ammonia, high nitrites, and high chlorine are considered to afford complete proof of pollution by sewage. Dr. Drown has pointed out, however, that when the chlorine is not much above the normal in waters containing high free ammonia and nitrites the inference is that the pollution comes from farmyards or manured fields, a distinction that it is often important to make.

Table No. 3, giving a number of analyses of waters, shows what is meant by high and low amounts of the various compounds referred to. It is taken from the Twenty-second Annual Report of the Massachusetts State Board of Health, and the figures stand for parts per 100,000. The first six analyses are of unpolluted waters, while the last five are of polluted waters.

Table No. 3.—Water Analyses.

	Free Ammonia.	Albuminoid Ammonia.	Nitrates.	Nitrites.	Chlorine.
1	0.0000	0.0022	0.0060	0.0000	0.08
2	0.0702	0.0030	0.0030	0.0006	0.10
3	0.0000	0.1252	0.0000	0.0000	0.10
4	0.0130	0.0333	0.0250	0.0001	0.16
5	0.0000	0.0136	0.0050	0.0000	0.63
6	0.0000	0.0152	0.0060	0.0001	2.10
7	0.0124	0.0284	0.0130	0.0000	0.19
8	0.0000	0.0196	0.0550	0.0004	0.54
9	0.0016	0.0198	0.0200	0.0004	0.58
10	0.0000	0.0262	0.0170	0.0010	2.09
11	0.0664	0.0263	0.0800	0.0025	2.41

The hardness of water is expressed in the analyses of the Massachusetts State Board of Health by comparing it to the amount of carbonate of lime that would have the same effect in an otherwise pure sample. A hardness of 2 means that the effect of the soap curdling substances in the sample would be produced by water containing 2 parts per 100,000 of carbonate of lime. This property has little hygienic signification, but it is important in other respects, for if the substances causing it are present in large amounts the water causes trouble in steam boilers. Frequent reference is made in reports and books on water supply to Dr. Clark's scale of hardness. In this scale each degree represents the hardness of water containing the equivalent of one grain of carbonate of lime to the imperial gallon; 22 degrees of hardness is therefore that of an imperial gallon of water containing soap-curdling matter producing the same effect in it as would 22 grains of lime. Waters of more than 5 degrees of this scale are usually considered

hard. Chemists distinguish between permanent and temporary hardness, the latter being the amount or number of degrees that may be removed by boiling. A water temporarily hard will give a sludge or mud in a boiler, which is easily cleaned out, while one permanently hard will produce the hard scale which boiler-tenders find so difficult to remove.

The chemist who makes an analysis of water should always be requested to supply an interpretation of it so that the results of his work may be understood by persons without chemical training. If his report is properly prepared the foregoing hints will probably enable its significance to be understood readily. The reports of bacteriological analyses should always be accompanied by a statement as to what they go to prove, as bacteriology is a science of such rapid progress to-day that only a specialist's interpretation of an analysis is of much value.

CHAPTER II.—EARTH DAMS.

An impounding reservoir is often regarded as nothing but an artificial lake, to be formed in the cheapest possible manner, but if the water is to be used for domestic purposes, such a view will be liable to lead to trouble. The dam itself, if poorly designed or built, may be a source of danger to people living below it; if it is not carried into the underlying strata and into the banks so as to prevent leakage, considerable water may be lost; if the surface soil is allowed to remain, the quality of the water may be very bad at times; if there are places along the borders of the reservoir where the depth of water is shallow, weeds may grow and injure the quality of the water when they decay; if the reservoir is so located that the disposition of the strata under and around it is unfavorable, the water may leak out and the reservoir prove little better than a strainer, provided the bottom is not made tight in some way. These are all important matters to be settled before beginning work, and the history of water-works construction in this country shows that much money has been wasted and much annoyance caused by failure to take them into consideration.

Fairly water-tight dams can be made of many materials which are unsuited for use in reservoir embankments. A manufacturing canal at Nashua, N. H., has a bank composed of nothing but pure sand, through which in its natural condition water passes freely. In the 60 years this canal has been in service the sand has become silted with fine material deposited from the water, and it is now practically tight. The old Middlesex Canal in Massachusetts had banks of loose gravel which must have allowed large quantities of water to pass early in its history, but the sediment in the water filled the interstices and finally made the banks tight. Reservoir embankments of sand are not uncommon in India, and the construction of one of the best of these is described very fully in the "Proceedings" of the Institution of Civil Engineers, Volume cxv., by Col. S. S. Jacob. According to Mr. Albert F. Noyes, M. Am.

Soc. C. E., a reservoir embankment about 16 feet high has been built of fine sand, the slopes being 1½ to 1 and covered with the turf taken from the ground. There was no appreciable leakage of water. A section of the bank was finally carried away, the break being caused by a woodchuck hole. It was repaired by filling in the opening with the same kind of material that was washed away, and then facing the inside slope with hardpan, a kind of clay gravel, put on in thin layers until a total thickness of 2 feet was obtained. All stones above 2 inches in diameter were picked out carefully, and the layers were compacted by spading rather than by rolling or tamping. This facing was covered with 4 inches of broken stone and a layer of riprap 12 to 14 inches thick. It was found best in making such an embankment to have the material slightly damp but not wet, and to use no water while rolling the layers. In spite of the favorable experience with some dams made of sand, it cannot be regarded as a material suitable for such works, when anything better can be secured at a not prohibitory cost.

When it comes to considering the other varieties of earth used for embankments, there is a very troublesome obstacle to collecting information—viz., the lack of uniformity in the use of the terms applied to the various materials. There are a few rule-of-thumb practices regarding earth in which technical terms are not involved, among them is one adopted in the neighborhood of Lowell, and described by Mr. Clemens Herschel, M. Am. Soc. C. E. There the fitness of a material for puddling, or making a water-tight coating or layer, was ordinarily tested by placing in a pail of water enough of it to render the water invisible. The pail was then turned upside down, and if the mixture dropped out it was rejected; if it remained in the pail it was considered satisfactory for puddling.

The importance of a proper selection of material for an embankment is so great that a number of opinions of leading American engineers are quoted verbatim below. The author believes that these opinions are of great practical value, and to make them as useful as possible they are classed under the general heads of clay, gravel, and mixtures.

CLAY.

"A good illustration of the behavior of clay when used for puddling material in such proportions that its removal from the mass

by water very seriously disintegrates the whole mass, occurred on the Ridgewood reservoir of the Brooklyn Water-Works, built in 1857-60. The reservoir and embankment were built of material taken from the excavation, and well rolled and rammed. This material was the Long Island drift. Where the reservoir was excavated below the natural surface of the ground the banks were dressed to a slope of 1½ to 1, and a puddle facing 18 inches thick at right angles to the slope was put on, consisting of clay and gravelly earth in about equal proportions. This material was wet and cut with spades. Above the natural surface the puddle wall was carried over the natural surface to the center of the embankment, and then carried up vertically in the center of the embankment. These banks were faced with a slope wall of split boulders about 12 inches thick, with the interstices well filled with pinners. When the reservoir was filled the water dissolved the clay out of the puddle, and the slope wall slid in some cases and settled back in others, and it was necessary to empty the reservoir and relay the whole of the wall in cement mortar. Since that time I have never attempted to use clay in puddling to which water could find access, and I think in general the less of such material there is used in puddling a wall the better the wall is."—J. J. R. Croes, M. Am. Soc. C. E.

"The particles of clay are cohesive, and a vein of water ever so small which finds a passage under or through clay is continually wearing a larger opening. An embankment of clay is much tighter at first (than one of gravel) but is always liable to breakage."—William J. McAlpine, Past President Am. Soc. C. E.

"Clay becomes slimy and sticky when wet, and yet it is difficult to mix it thoroughly with water. Hence voids are apt to occur in the body of the puddled mass. As the water leaves it it shrinks and cracks, and yet retains water in parts; so that it never properly settles down and becomes compact, but is liable to be cut away if only a small stream of water passes through it, or if it is placed in water which is only gently agitated. I have no doubt that clay owes its reputation in this country to its mention in accounts of English work. There is a vast amount of what farmers call 'in-and-in-breeding' in the education and training of water-works engineers in the United States; so that when an error of this sort is once engrafted it is not easily eradicated.—Clemens Herschel, M. Am. Soc. C. E.

"Some clays are apt to become saturated with water and under certain conditions to become fissured. They cannot, therefore, be used alone. Moreover, unless a clay is exceptionally tough, an aperture through it, however minute, is apt to become enlarged and finally to cause serious trouble. We find, however, that a number of dams of great height are reported from California as being built of clay. The designer of several of these dams stated that he had subjected a cubic foot of the clay to a hydraulic pressure much superior to that corresponding to the expected depth of water behind it and had been unable to force water through it; but these clays must be of very exceptional quality."—A. Fteley, M. Am. Soc. C. E.

"Fat or unctuous clays are mostly designated by writers on hydraulics as the only proper material of which to form an impervious, water-tight wall in the heart of an embankment, or the entire mound of a reservoir, as the case may be; and yet I am convinced that more failures of reservoir embankments and of high earth dams are due to the too free use of pure clay in puddled core walls, and to the almost entire dependence placed upon such walls, than to any other cause. The mistake too often made by engineers is that of supposing that only clay can be used for puddling. An embankment built entirely of clay is an unsafe one, even when puddled in the very best manner possible. It is easily attacked by muskrats and by other foes of a water-tight embankment."—E. F. Smith, M. Am. Soc. C. E.

GRAVEL.

"At many places the word gravel is understood to mean a mass of rounded stone of varying sizes. This sort of gravel occurs in very large deposits in many places and it is similar in the form of its constituent parts to the gravel of the seashore. The other sort of gravel is made up of stone not rounded, but rather of flat shape, and with many particles of very small size, but still not rounded. It is with this latter sort of gravel that tight puddle can be made without the admixture of clay. The gravel composed entirely of rounded pebbles of varying sizes will not alone make a tight bank, but in many cases where clay occurs and only this sort of gravel can be found, an excellent puddle is made by a suitable mixture of the two. This is the case in the banks of the canals of the State of

New York, at many points in which a section of such puddled material is formed in the center of the bank, and which, when cut into afterwards, is found to be compact and impervious."—John Bogart, M. Am. Soc. C. E.

"By the term gravel is not meant a collection of clean, round, water-washed pebbles of nearly uniform size, but a combination of small stones, sand, and loam so proportioned that all interstices between the stones will be thoroughly filled by finer materials. In certain proportions clay is valuable in such a mass, provided that it is so situated that water cannot get at it so as to wash out the clay, which consists of very fine particles capable of being washed away by the action of running water."—J. J. R. Croes, M. Am. Soc. C. E.

"The particles of fine gravel have no cohesion. A vein of water first washes out from the gravel the fine particles of sand, and the larger particles fall into the space, and these small stones first intercept the coarser sand and next the particles of loam which are drifted in by the current of water, and thus the whole mass puddles itself better than the engineer could do with his own hands. The vacuities produced below by this operation are indicated by the settlement at the top, where more gravel, etc., can be added as is found necessary. An embankment of gravel is comparatively safe and becomes tighter every day."—William J. McAlpine, Past President Am. Soc. C. E.

"Gravel capable of being puddled will do anything that clay was ever used for in water-works practice, and will do it better. I have known cases where clay was brought at considerable expense to a bridge site to be filled into bags and used in coffer-dams, while good gravel, which would have done the work much better, abounded close at hand. Clay placed in such bags washes out and disappears, while gravel retains nearly its full dimensions in water."—Clemens Herschel, M. Am. Soc. C. E.

"Gravel suitable for use as a reservoir embankment may be defined as a material resulting from the disintegration of any of the harder rocks, with the admixture of water-washed pebbles and stones not larger than pigeon eggs nor smaller than the grains of coarse sand, with sufficient clay to bind the mass together when compressed. The presence of a suitable binder in the form of clay is the one important element in the make-up of gravel suitable for puddling."—E. F. Smith, M. Am. Soc. C. E.

MIXTURES.

"The material in this section (Pittsburg, Pa.) contains more clay than does that found in Massachusetts, and when it is excavated after a long dry spell, it is apt to come up in large lumps. If placed on the banks in thin layers in its natural condition and thoroughly rolled so as to pulverize the lumps, the absorption of water upon the filling of the reservoir causes the material to swell, and in my opinion, to make a tighter bank than when the material is put in wet."—James H. Harlow, M. Am. Soc. C. E.

"The writer is of the opinion that clay, on account of the fineness of its particles and of what is commonly called its binding qualities, must enter into the composition of the material used. That a very small proportion of it is sufficient is shown by the very excellent behavior of banks wholly formed of hardpan, in which the gravel and fine sand are cemented by the admixture of various proportions of clay."—Alphonse Fteley, M. Am. Soc. C. E.

"When engineers appreciate the fact that a homogeneous bank of gravel, compacted by a little clay, is better than a clay core with indifferent material on both sides of it, the number of failures will be comparatively few. Where true clays are used in proper proportions with other material they are fitted for the purpose of reservoir construction. In this I do not include those so-called clays which originate from sand that has been reduced to finely rounded grains and which rather resemble quicksand."—E. F. Smith, M. Am. Soc. C. E.

"The best natural puddle we have is hardpan, and if any of you have worked in hardpan you will have noticed that a great deal of gravel is encountered in it; it is also pretty hard to work, while pure clay can be easily worked. My experience has led me to work about 3 to 1—that is, I take pure clay, cut it up, and to every three barrows of it dumped down an embankment, or anything of that kind, I dump down a barrow of gravel and sand."—Robert Cartwright, M. Am. Soc. C. E.

These quotations furnish such explicit information on the subject of earths that little further comment is necessary. It is frequently desirable to form artificial mixtures of various materials in order to make certain portions of a dam particularly tight. The experienced engineer can generally decide upon the proper proportions without recourse to trial, but in case experiments are

necessary, the method recommended by Col. John T. Fanning, M. Am. Soc. C. E., is probably as expeditious and satisfactory as any. It consists in filling a water-tight box with the coarsest material available, and then pouring in water until all the voids are filled. The water is drawn off and measured, showing the amount of empty space in the box. Then this material is mixed with as much of the next finer available grade as it is possible to get into the box, and the voids measured with water again. When this process has been repeated down to the mixture with sand, the voids then left unfilled are to be filled with clay. Colonel Fanning states that if such an experiment is begun with 1 cubic yard of coarse gravel, and the mixing of the materials is well done, 0.28 yard of fine gravel, 0.08 yard of sand, and 0.03 yard of clay will make a mixture having voids of microscopic dimensions. As a standard for practical purposes he recommends 1 cubic yard of coarse gravel, 0.33 yard of fine gravel, 0.15 yard of sand, and 0.2 yard of clay.

MASONRY CORE WALLS.

There is probably as much controversial literature on the subject of masonry core walls in the center of dams as on any subject pertaining to water-works construction, but it certainly is true that the general trend of opinion is in favor of their use in impounding dams. When properly built they undoubtedly add to the safety of an embankment. Unfortunately they have sometimes been poorly built and badly designed, and have naturally proved useless or worse. It has been stated by one eminent American engineer that a core wall of one dam that failed was built of such insufficient dimensions that a horizontal section formed a convenient paper-weight, the thickness of the wall being 6 inches. On this subject the opinions of a number of prominent engineers are presented in the following quotations:

"It seems to me that a core wall is always a good thing, and that in almost any ordinary case it is worth its cost. There are cases, of course, where its cost would be prohibitive, and there are cases where the material is of such an excellent nature that one is justified in getting along without it. But I will confess myself to having a very strong preference for a core wall, both as a preventive against the workings of muskrats and of woodchucks and as a means of making the embankment tight; and also as a means of causing the destruction of the embankment to take place

gradually in case the water does penetrate it."—John R. Freeman, M. Am. Soc. C. E.

"As I said before, my idea of an ideal reservoir would be one of solid masonry, and it can be made tight, and when it is once made tight it is there forever. But we can't always afford to build a reservoir of solid masonry, and the next thing to it is to have a core wall of masonry, which I should construct every time when I could get the material."—M. M. Tidd, M. Am. Soc. C. E.

"In high reservoir embankments of the *fin de siecle* American pattern, sheet piling is replaced by a core wall of either concrete or masonry, founded upon the ledge or upon some other trustworthy substratum below the original meadow level, and extending up to the full-water level. This core is made some 4 or 5 feet thick at the top and at the bottom, tapering each way from top and bottom to a thickness of 7 or 8 feet at the original meadow level. I have heard these core walls criticised on theoretic grounds, but I know of no valid objection to them."—Clemens Herschel, M. Am. Soc. C. E.

"When it can be done within proper limits of economical construction, the writer prefers to secure water-tightness by means of an impervious wall built in or near the center of the embankment and continuously connected with the impervious bottom or extended downwards to a safe depth. There is no question that homogeneity in the mass of an embankment is very desirable; but the writer, with some experience of the difficulty of obtaining perfect work at all times, and of the trifling causes that can produce a leak through an earth embankment, prefers to use a masonry wall as a core. If a small defect exists in the core wall, only a limited amount of water can find its way out. I do not see that the presence of a masonry core wall weakens the structure in which it is built, for the tendency of the earth is to settle against the masonry. At any rate, I have failed to find, in all the cases of accidents which have come to my knowledge, any that were due to the presence of a core wall. The cause of the failure was invariably a defect in design or in construction."—Alphonse Fteley, M. Am. Soc. C. E.

"With regard to the construction of earthen dams, the speaker stated his belief that an impervious core wall could be obtained of other materials than masonry which would be just as good; it must be made of the best materials, however, and well placed.

Many miles of embankments and dams are now standing in the United States in which the center is a wall of puddled earth and clay, combined in suitable proportions and placed properly, and such walls are as impervious as any of masonry can be."—Discussion by J. J. R. Croes in "Transactions" Am. Soc. C. E.

"The speaker did not wish to go on record as being opposed to a masonry core, and he had under consideration at that moment a dam in which he was strongly inclined to put one. On the other hand, he regarded such a core as an extreme precaution for the sake of safety. If he had just the material wanted and was certain that water would not rise to the top of the dam he was designing, he might be willing to build it without any masonry core, although it is 60 feet high."—Discussion by John Bogart in "Transactions" Am. Soc. C. E.

Where the masonry core wall is omitted, and the material of the dam is not exceptionally water-tight, it is necessary, in the class of dams under consideration, to make a core of puddled earth or a facing of that material on the upstream slope of the embankment. In most cases the former construction is preferable for the reason stated concisely by Mr. Fitzgerald, as follows: "For the same amount of money more puddle can be put in a vertical wall than on a slope of equal height. A puddle wall in the center of a bank is not exposed to the danger of slipping when the surface of the reservoir is suddenly drawn down. The water does not seem to work out of the puddle (when on the face of an embankment) quickly enough to drain the bank, and the result is a head, which caves the slope. Puddle in the center of an embankment is less exposed to frost, to drying out and to cracking than it is on the slope." The puddle earth ought to be of the best quality, free from lumps, without any stones larger than 2 inches, and preferably still smaller, put on in layers not exceeding 6 inches in thickness, and cut and crosscut with spades every inch or so, until the layer is not more than 4½ or 5 inches thick. Where the puddle covers such an area that grooved rollers can be employed to advantage, it may be used in 4-inch layers and compacted in the manner to be described under the head of methods of construction. There is no question that such a mass of puddle, if made of small, clean gravel with some clay, will be practically water-tight, but it cannot be relied upon to stop the burrowing of muskrats and woodchucks, which only masonry will prevent.

No matter what material is employed for a core wall, it must be carried down far enough and deep enough into the sides of the valley to prevent the passage of water under or around the dam through the undisturbed natural earth. This subject will be discussed later.

WATER IN EARTHWORKS.

The tendency to use too much water in consolidating the layers of earth in an embankment is perfectly natural from a contractor's point of view, because the liberal use of water will apparently make a very tight bank with the minimum expenditure of labor and time. With most of the materials used for such works, however, there is a strong probability of chinking of the bank as the surplus water dries out, if an excessive amount is used. Everyone has noticed how mud flats crack when exposed to the sun, and the action is the same, though on a much smaller scale, in an embankment kept too wet during construction. An experimental proof of this was furnished by some investigations made by Mr. FitzGerald in connection with his studies of evaporation. He filled eight large zinc-lined tanks with gravel, sand, earth, and mixtures of puddle and other materials. In all cases where the filling was wet it was found to be impossible to keep the tanks full, no matter how much ramming was done. By ramming in dry materials, however, no trouble from shrinkage was experienced, and in every case the material brought the paint off from the zinc when it was removed. Sometimes, as in the case of the reservoir of the new water-works at Astoria, Ore., the earth for a reservoir embankment is found in such a condition that no watering at all is necessary. With hardpan, however, some water must be used to make a compact bank. This material breaks up in lumps about 3 to 6 inches in size, which it is difficult to pulverize, especially if hard. If a little water is used to moisten the lumps, and the moist material is laid in courses not over 4 inches thick and well rolled, the bank can be made very tight. Dexter Brackett gives the cost of the watering and rolling of an embankment containing 80,000 cubic yards of hardpan as about 2 cents a cubic yard, the work being done thoroughly. It is particularly important to have the rolling and ramming of the material done under the supervision of a conscientious inspector, where but little watering is done, as the work is apt to be slighted. Alphonse Fteley advises the use of enough water "to give plas-

ticity to the earth and to moisten it to about the same degree as is observed in a deep excavation free from water."

In the construction of the Clinton and Oak Ridge dams of the East Jersey Water-Works the gravel composing them was spread in thinner layers than usual, not over 4 inches thick, which were reduced to about 2½ inches after rolling. J. Waldo Smith, M. Am. Soc. C. E., reports that at first the method of sprinkling and then rolling was followed. It was found that better work could be done and the material compacted more thoroughly by rolling first, then sprinkling and rolling a short time, and giving a final wetting just before the next layer was applied. The theory on which this procedure is based is that the air can be better forced out of the loose gravel, and the material made more compact while in a dry or naturally moist state, and that the subsequent addition of water still further settles and binds the whole mass. Whatever merit this theory may have, the practice it leads to, the use of a minimum quantity of water, agrees with the best usage at present.

The following clause is taken from the specifications for Reservoir M of the Croton Water-Works of New York City:

"Ample means shall be provided for watering the banks, and any portion of the embankment to which a layer is being applied shall be so wet, when required, that water will stand on the surface. The contractor shall furnish at his own cost the necessary steam or other power for forcing the water upon the bank if the engineer finds that other means of transportation and distribution of the water are not sufficient."

The specifications for the dam for Reservoir No. 5 of the Boston Water-Works read much the same on the subject of watering. The wording of the clause in them is as follows:

"Ample means shall be provided for watering the banks, and any portion of the embankment to which a layer is being applied shall be so wet, when required, that water will stand on the surface. The contractor shall furnish at his own cost the necessary steam-pumping plant and force main for forcing water into a tank situated on the side hill, at least 50 feet above the top of the dam when completed. From this tank a 3-inch distribution pipe, with gates and hose connections, will lead lengthwise over the dam to supply water wherever it may be needed. If the engineer approves, some other method of equal efficiency for the furnishing of water may be substituted for the above plant."

These clauses refer to particular pieces of work very carefully studied by Messrs. Fteley and FitzGerald respectively. While they might be considered as indicating a preference for heavy watering, if they are examined carefully in connection with the previously mentioned statements of these engineers, it will be apparent that they are intended mainly to secure provision by the contractor of ample watering facilities, in case they should be needed. Like all other clauses from specifications which are printed in this series of articles, they are to be regarded as suggestions only, and not copied verbatim. It would be very foolish to insert the Boston clause in specifications for a small dam to be made of a naturally moist mixture of gravel, sand, and clay which does not require watering.

CROSS-SECTION OF THE EMBANKMENT.

The cross-section given the dam depends somewhat on its height, but the general form has been pretty well standardized by this time. American engineers have learned that it is unnecessary to have such flat slopes as are usually adopted abroad; 2 to 1 on the inner face, and the same rate, or 2½ to 1 on the outer face, are the slopes generally employed. If the dam is quite high, a berm 5 or 6 feet wide should be made on the inner face a few feet below the low-water level, and a wider berm should be built on the outer face about one-third to one-half the distance from the top. The latter was probably first used in this country by Mr. FitzGerald, and provides a means of longitudinal drainage to protect the loam covering of the slope until the grass becomes well rooted on it. This berm must be well drained itself, however, or water will pass from it under the grass, undermining and destroying it. The height of the dam above the high-water level depends somewhat upon the size of the structure. If it is a high dam, the bank should rise above this level about 18 inches plus the depth of the frost in the locality where it is built. If the structure is low, this height is often made less and reliance placed on additional width at the top to prevent frost cracks from opening passages for the passage of water. Allowance must also be made for the height of waves in some cases. The width of the dam on top should not be less than 10 feet unless the structure is very small. All angles in the cross-section as first sketched out should be rounded off, as angles in earthwork are entirely theoretical, except in rare cases.

STARTING THE CORE WALL.

The portion of the dam liable to give the most trouble is the bottom of the core wall or the puddle core, since it has to be carried down to impervious material and may require difficult trenching in water bearing strata. Quicksand is usually the worst material to deal with, and in case it is encountered the following notes may prove of value. They are abridged from a very valuable paper on the method in which a breach was repaired in the dam of the storage reservoir of the New Bedford Water-Works. This paper was written for the American Society of Civil Engineers by the late William McAlpine.

In the course of the repairs, a trench had to be made water-tight with sheet piling exposed in places to a head of more than 20 feet. For such a purpose Mr. McAlpine considered driven sheet piling worse than useless. His plan was to excavate the trench to the greatest depth practicable, and to place in the bottom a horizontal timber to which closely jointed planks were spiked. These planks were spiked to a similar timber at the top, and were covered by a second course of jointed boards. A water-tight barrier can be made in this way, which it is very difficult, if not impracticable, to obtain with driven plank. Another point to be considered in sheeting a trench is that water will pass horizontally and vertically along a smooth surface for a great distance until it finds open joints through which it will pass freely. Water abhors angles, and by compelling it to make a sufficient number, its head can be destroyed entirely. The interposition of angles is often the only defense the engineer has against water, and the practice of placing instead of driving sheet piling enables many of these to be obtained. There are three rules to be observed in excavating quicksand. (1) the water must be removed promptly and thoroughly, (2) the excavation must be made with the utmost dispatch, (3) the material must not be disturbed after it begins to quake. Quicksand is a mixture of fine sand with such a proportion of clay or loam as enables the mass to retain water within itself; and when in this condition, after it has been trampled upon for a short time, it begins to quake, so that it may also be called quakesand. When it reaches this condition, if it is left quiet for a few hours, the heavier particles of sand and clay settle down and expel the water, and the mass becomes firm again. If, on the

other hand, it is further disturbed by the feet of the workmen, it becomes more and more fluid, additional material flows in from the sides, and no progress can be made in the excavation. When the engineer has such a work on hand he should provide ample pumping power, and in most cases he will find a power several times as great as he anticipated will often prove most economical in the end. The pumps should be capable of lifting sand as well as water, and those are best which are not liable to be clogged; this is of more consequence than that they should work with a good duty.

Mr. McAlpine found that in most cases sheet-piling protections around the pit to prevent the influx of sand are useless and often detrimental. If there is room to allow the excavation to take its natural slope and the three rules are observed, the sheet-piling protection will be found unnecessary. Quicksand in a dry state may be excavated nearly vertical.

In the work at New Bedford the dimensions of the pit on top were about 50 feet width by 100 feet length. There were 30 laborers employed. Six were kept constantly at work removing and casting aside the sand from about and under the pump, to keep it far below the other parts of the excavation. Twelve men were employed all the time opening small ditches radiating from the pump pit. Six men were employed in excavating the ridges left between the radiating ditches, and as long as the latter were kept open these ridges offered perfectly dry digging. The remaining men were employed in casting further back the earth which was thrown out by the six men last mentioned. The actual removal of the earth was measured by the work of only six men out of 30, but these six had perfectly dry earth to handle. There was considerable difficulty in compelling the men to follow the rules mentioned previously. They were violating them constantly, although they had palpable evidence of the disastrous results due to their neglect.

Work was begun early in the morning, and by noon the pit had been sunk to a depth of 12 feet at the lowest place. By this time it was apparent that the extreme capacity of the pump had been reached, and it became impossible to keep open the radiating ditches. Consequently the earth between them became suffused, and the whole of the lower portion was transformed from hard compact sand into a semi-fluid material quaking like jelly. Water

began to boil up in many places in the bottom, and it was evident that no further progress could be made at this time.

The method of carrying on the work was then changed. It was very important that sheet piling should be placed at a much greater depth than the excavation had been carried, but Mr. McAlpine believed that to drive the plank to the desired depth would have resulted in open joints at the bottom. It was therefore determined to lessen the number of such joints by making up the plank in panels of 4 feet width, with their joints matched and battened with 1-inch boards. One of these panels was placed in the proper line of the sheet piling, and forced down by pressure to a depth of nearly 5 feet. A second panel was forced down in the same manner and with considerable trouble a tolerably close joint was made with the first and further secured by a plank driven over the joint. Successive panels were placed in this manner until the pit was covered by two rows of sheet piling placed 15 feet apart. There was considerable leakage through this piling, but by spending about $200 in improving the sluice by which the water in the river was conducted past the pit, more than half the leakage was prevented. Under the new conditions the pump was found to have ample power to free the pit from water and enable the joints in the sheet piling to be calked so that the puddle could be put in.

The full account of the work thus carried out will be found in Paper No. VII. of the "Transactions" of the American Society of Civil Engineers. It is one of the most helpful articles ever written for engineers engaged in hydraulic work, and is fortunately still in print.

Another method of dealing with quicksand which has proved very satisfactory in many places consists in sinking driven wells along both sides of the trench. By pumping continually from these wells the subsurface water is kept at a low level in the vicinity of the construction. An illustrated description of the method followed in such a piece of work on the Metropolitan sewerage system of Boston was published in "The Engineering Record" of January 21, 1893.

CHAPTER III.—MINOR DETAILS OF RESERVOIRS.

For the class of reservoirs used for small works such as are described in these articles, the construction of the pipes, gate-houses and waste weir is a very simple matter. But two pipes will be needed in the great majority of such reservoirs, one for the regular supply and the other for a waste pipe through which to draw off the impounded water. In carrying a pipe through an embankment subjected to water pressure, there are two points which must always be kept in mind. The first is that the exterior surface of a pipe or conduit of masonry offers excellent opportunities for leakage of water, and the second is that a pipe carried through an artificial bank is always exposed to breakage by the settlement of the earth.

Mr. McAlpine's famous axiom, "Water abhors an angle," has already been mentioned, but it may be repeated again here as indicating the method of preventing the leakage of water along the pipes. The greatest care should be exercised in laying the pipes to have the earth tamped firmly against them, but even the most painstaking supervision in this respect will fail to prevent the passage of water along them. Hence cut-off walls of concrete or good masonry should always be built around the pipe to break the uniformity of surface and prevent the percolation of water. The late M. M. Tidd was accustomed to have cast on some of his pipes to be laid through an embankment, a collar or flange, 2 or 3 inches high and perhaps 2 inches wide, which was firmly bedded in cement. In small reservoir dams a masonry cut-off placed around the pipe midway between the core wall and the foot of the inner slope, and another midway between the core wall and the foot of the outer slope, will be sufficient to stop any leakage, if the earth is tamped firmly about the pipe. These blocks of masonry should be made with great care, and if they stand out 18 to 24 inches from the pipe no harm will be done; they need not be more than 18 to 24 inches thick.

When a pipe is carried through an embankment it ought never to be supported on a series of masonry piers, one under each joint, as this practice is simply an urgent invitation to troubles of various sorts. In case the embankment settles, every length of pipe is subjected to a bending stress which tends to cause leaks. The moment a leak occurs, the water under pressure enters the earth about the pipe and loosens it, and if the water does not sooner or later pass along the outside of the pipe to the face of the dam and cause a washout of part of the structure, it will be more a matter of good luck than good engineering. The fact that many pipes supported in this way have not broken does not make the practice a good one. The ideal manner of carrying a pipe through a dam is by resting it on some natural foundation, ledge, or dense hardpan, which is known to be absolutely unyielding, but if this cannot be done, a concrete or cement masonry wall, with several projecting cut-offs and a rough surface, ought to be built to support its entire length. The dimensions of such a foundation wall depend, of course, on the local conditions of each problem.

The ideal method of leading pipes from a reservoir is to carry them through the natural foundations at one side of the embankment. This is generally quite expensive and does not offer many advantages in the case of small reservoirs, although where the dams are high this plan is to be selected, if possible, for reasons which it is unnecessary to explain here.

Too much emphasis cannot be laid on the importance, in any dam having pipes laid through it, of preventing the percolation of water along their exterior surface, and of preventing the breakage of the pipes by settlement.

GATE-HOUSES.

The gate-house of a small reservoir is a very simple structure. In designing it, care should be taken that it is firmly founded, and that there is no chance for the pipes from it to become broken. The nature of the foundation is naturally governed by local conditions, but in case it must be on any other material than rock particular care must be paid to securing an ample bearing area, as the bottom of the gate-house and its connection with the dam are frequently weak points in an otherwise good design. There should be an opening in the wall at the level of the bottom of the gate chamber to enable the water to be drawn

off to the bottom of the reservoir, and it is a good plan to cover the bottom of the reservoir with riprap in front of the entrance, if this can be done without much expense. The other openings for water may be from 6 to 8 feet apart vertically. The size of the openings at each level may be determined by means of the following formula: $A = 0.3\ Q$, where A is the area in square inches and Q is the maximum quantity of water in cubic feet per second which the pipe line from the reservoir will carry.

Manufacturers of valves now supply sluice gates for any openings which will be required for small gate-houses. They have bronze facing on the seats and gate faces, and can be had with a spigot on the back for building them into the wall or with flanges by which they may be bolted to the end of the pipe. The gates are raised in two ways, by means of a rising stem and by means of a non-rising stem. The rising stem is a rod firmly attached to the gate and threaded at the upper end. A handwheel bearing against a large washer or plate is screwed on the end of the rod, and by turning the wheel the gate is manipulated. The non-rising stem has a thread at the lower end and is keyed to the handwheel. The gate has a threaded hole through which the lower end of the rod passes, and is raised and lowered by turning the rod by means of the wheel. The rising stem is cheaper and preferable in most cases. The face of the sluice opening on the outside of the gate-house should be finished off smoothly so that it may be closed with a wooden plug wrapped with canvas in case of an emergency.

The gate chamber is generally built of rubble or brick masonry, but sometimes other materials have been used. Now that vitrified brick are so cheap the best lining in many cases would be made of this material, but good hard-burned brick will answer. In fact no lining at all will be needed with the rubble masonry of some parts of the country, as a really good Portland cement finish will answer all purposes. This applies to small work only. In the design of the gate-house particular care should be paid to securing a structure which will not be exposed to damage by the ice. During the winter and early spring there is always a possibility that the water will rise, lifting the ice and loosening that portion between the gate-house and the sloping back of the dam, so as to bring a strong pressure against the former. In very cold climates there is also danger of the ice lifting the masonry if the latter is

very light. For these reasons a gate-house is generally a more massive structure than a statical analysis of the strains upon it would call for.

The arrangement of the effluent pipes from a small reservoir admits of little variety. Two different plans are shown in Figures 2 and 3, and modifications of these will probably answer all purposes. The first plan, shown in Figure 2, will answer for the

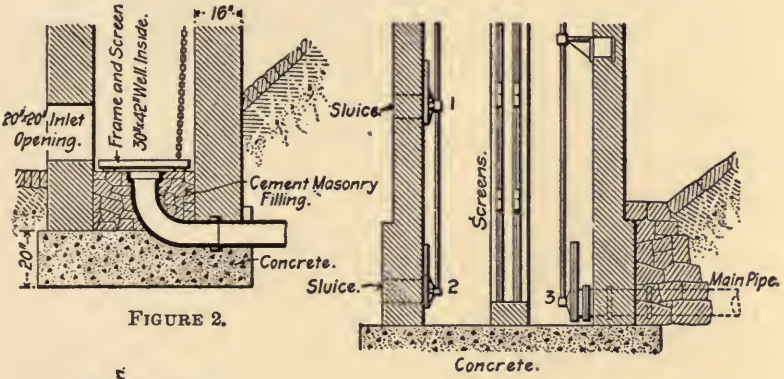


FIGURE 2.

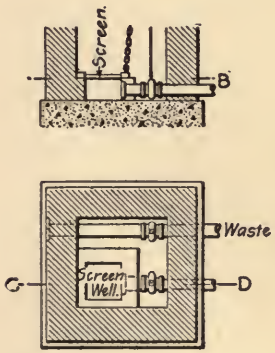


FIGURE 3.

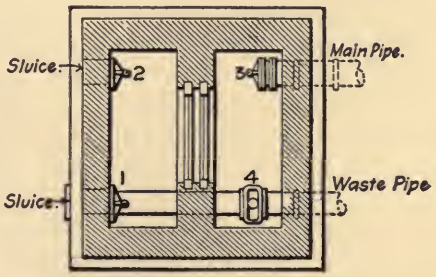


FIGURE 4.

smallest reservoirs. The waste and main pipes end in elbows, with the openings flush with the bottom of the chamber, and have no valves at the inlet end. The cut is a section through one of the pipes. In case it is desired to close the end of either pipe, it can be done very quickly by means of an iron plug wrapped with canvas, a little hemp being placed as a padding between the can-

vas and iron. The tightness of such a plug is surprising, and it can be placed in position by means of a rope or chain when the well is full of water. The suction of the water entering the pipe will seat the plug firmly. Above the mouths of the pipes is a weighted wooden frame having a screen of No. 10 copper wire making quarter-inch meshes, or a sheet of copper perforated with quarter-inch holes close together. The copper sheet is cleaned more easily than the screen. The frame is arranged so that it can be lifted to the surface readily. When such a system as this is employed a small valve chamber should be built at the outer toe of the embankment and covered with a strong wooden or masonry box, by which the valves may be reached. The cover to the box should be provided with a good lock to prevent any tampering with the valves. The waste pipe should be carried far enough beyond the gate-house to reach a convenient locality for discharging readily the water passing through it. The end of the pipe should be protected in many cases by a cement masonry wall resting on a good foundation, and the ground immediately in front of the end of the pipe should be paved with small field stone to prevent the washing away of the earth. Such an arrangement as this is probably as cheap as anything reliable that can be designed. The extra cement masonry filling required to form a level surface at the mouth of the elbow and for the small valve chamber will be less expensive in most cases than the standards and fittings necessary to manipulate the valves were they placed in the main well instead of the small gate chamber. Such a system is to be selected for the smallest works only, where it is necessary to keep the first cost down to the minimum amount. The greatest depth of water for which the plan is suited is not much over 10 feet.

A somewhat more elaborate gate-house is shown in Figure 3, where the valve chamber is 4 feet square, inside dimensions, and the walls are 16 inches thick. The two pipes, assumed to be 8 inches in diameter, are each provided with a single gate valve. The waste pipe runs directly from the outside face of the wall and has no connection with the valve chamber. The main pipe ends in a small screening well about 20 inches square and 1 foot deep. This is covered by a frame having a screen of wire or perforated plate. This frame can be pulled to the surface whenever it is necessary to clean it, and can be placed in position again without much trouble with the aid of a pole kept in the valve chamber.

With this plan it is proposed to draw water from the reservoir at different heights through sluice gates arranged in the manner shown in Figure 4.

The advantage of this method of construction over the first is due to the complete control of the water at the place where it leaves the reservoir instead of below the dam. The screen is horizontal and therefore more liable to become clogged than if it were placed in an upright position, but on the other hand the cost of such screening apparatus is very small as compared with that of the screens shown in Figure 4. The valves have bell ends and are calked to the pipe in the usual manner for such valves. In the cuts the vertical cross-section is taken on the line C D and the plan on the line A B.

The gate-house shown in Figure 4 is representative of the best class of such structures for small works. It was built recently at Ipswich, Mass., from plans prepared by Freeman C. Coffin, M. Am. Soc. C. E. The depth of water in this reservoir above the concrete foundation is about 18 feet, and two sluice gates are provided by which the supply can be drawn off of the bottom of the reservoir or from about mid-depth. The chamber is divided into two portions, each 8 feet by 3 feet 2 inches in plan, by the double set of screens and the grooved masonry walls in which they slide up and down. These screens have wooden frames measuring about $4\frac{2}{3}$ x 4 feet, four frames being used for each vertical set. The advantage of the duplicate screens is that when one set is removed for cleaning, the second set is still in place to prevent the entrance of large solids into the main pipe. The discharge pipe, it will be noticed, has no communication with the gate chamber in which its valve is located. The flow of water into the main pipe is controlled by a sluice gate with a flange connection, by which it is bolted to the end of the pipe. The top of the gate chamber is provided with a brick arch carrying the floor, which has a trap door through which the screens can be raised or lowered. The gangway to the gate-house from the embankment is carried by two 6-inch 16-pound I beams, and has a light railing made of 1-inch and $1\frac{1}{4}$-inch gas pipe. The bank end of the gangway is supported by two 6-inch vertical pipes resting on blocks of concrete imbedded in the dam. An 8-inch channel iron is laid over the top of the two pipes, and the beams, channel, and pipes are united by a few rivets and pieces of angle iron.

WASTE WEIRS.

The safety of an earthen dam depends in a great measure on the proper proportioning and construction of the waste weir by which the surplus water in the reservoir may be discharged. The first step in designing such a work is to ascertain the probable maximum run-off of the watershed above the dam and prepare the plans so that this entire volume may be discharged without allowing the high-water level in the reservoir to rise above the elevation assumed in designing the embankment. An examination of the site of the reservoir will often furnish indications of the great floods in the stream to be impounded, and valuable information can generally be secured from the residents in the vicinity. It is particularly important to remember that the discharge per square mile of small watersheds is liable to be excessive when compared with that of larger areas. Capt. James L. Lusk, of the Corps of Engineers, United States Army, recently made a valuable compilation of the freshet discharges from watersheds of moderate area; this is not easily accessible, so the most important figures are reproduced in Table No. 4.

Table No. 4.—Freshet Discharges in Cubic Feet per Second per Square Mile from Small Watersheds.

Watershed.	Year.	Area.	Discharge.
South Branch, N. Y.	1869	7.8	73.92
Woodhull reservoir, N. Y.	1869	9.4	77.76
Stony Brook, Mass.	1886	12.7	121.03
West Branch, Croton River, N. Y.	1874	20.4	54.43
Watuppa Lake, Mass.	1875	28.5	72.00
South Fork Creek, Pa.	1889	48.6	215.11
Flat River, R. I.	1843	61.0	120.85
Sudbury River, Mass.	1886	75.2	44.26
Rock Creek, D. C.	1856	77.1	126.40

Additional information bearing on the subject can be gathered from statistics of rainfall in the vicinity, if such have been kept; it is surprising how many amateur meteorologists there are throughout the country, and a diligent inquiry will generally secure some valuable information on heavy storms. Such figures, however, must be regarded as guides rather than as limits, for the waste weir must be large enough to discharge a greater quantity than has ever been measured, or the probabilities are that sooner or later the dam will be overtopped. In reporting on this matter to the Boston Water Board, the late James B. Francis, whose knowledge of rainfall and stream flow was remarkable, advised

proportioning the arrangements for the discharge of surplus water so that their capacity would be equivalent to a rainfall of 6 inches in depth in 24 hours over the whole watershed. This is several times the largest measured rainfall. Mr. Fteley made the same recommendation in a report to the Aqueduct Commissioners of New York City. His report reads as follows:

"As to the capacity of the overflow, it is necessary to depart from precedents on account of the extent of the watershed and the comparatively heavy rainfalls that occasionally occur in the Croton basin. Judging from the possibilities of rain or thaw in this and neighboring watersheds, the flowing capacity of the overflow should not be less than equivalent to the flow in 24 hours of a volume of water represented by a uniform thickness of 6 inches over the whole watershed.

"It is true that no such flow is on record, and the actual flow may never, it is hoped, come to that figure, but a combination of adverse circumstances, such as an exceptionally heavy rainfall occurring at a time when the ground is covered with snow, can bring about such a condition of things, and it is wise to be prepared for it. It can be so much more readily done that an increase in the length of the overflow can be obtained at a comparatively small cost.

"The writer having had occasion to design the overflow of several dams on an equivalent basis, may be allowed to state that, on the occurrence of a freshet which produced a flow somewhat less than one-half of the quantity just mentioned, he could not but feel in accordance with the sentiment of the people living lower down in the valley, that the channels of discharge were none too large."

Many attempts have been made to formulate a mathematical expression for the discharge from a watershed, but no one expression has yet been obtained which will give more than approximate indications. India has been particularly prolific in producing these formulas, and Mr. H. M. Wilson, M. Am. Soc. C. E., states the following are two of those most used in that country:

$$\text{Ryves' formula, } D = c\sqrt[3]{A^2}$$

$$\text{Dickens' formula, } D = c\sqrt[4]{A^3}$$

where A is the area of the catchment basin in square miles; c is a coefficient depending for its value upon rainfall, slope, soil, and

other local conditions; and D is the discharge in cubic feet per second. In the Dickens formula, the value of c for places where the maximum rainfall in 24 hours is 3.5 to 4 inches varies from 200 for flat country to 300 for hill country. Where the maximum rainfalls is 6 inches the coefficient ranges from 300 to 350. For the Ryves formula, the coefficient varies between 400 and 500, and for very hilly areas, where the maximum rainfall is high, it may reach as high as 650. Col. J. T. Fanning, M. Am. Soc. C. E., has given the following formula as an approximate expression of the mean maximum discharge from a number of American watersheds:

$$D = 200 \sqrt[6]{A^5}$$

Although such formulas have been employed to a considerable extent in the past, they are really of little value except as checks on estimates of flood discharge obtained in other more reliable ways, and in designing waste weirs, if the estimates exceed the results given by the formulas they should certainly be used. Mr. Desmond FitzGerald makes this very important statement concerning New England watersheds in his able paper on "Rainfall, Flow of Streams and Storage," previously referred to: "The waterworks engineer who is constantly designing waste weirs, dams, reservoirs, etc., may find it convenient to bear in mind that 1 square mile of land surface yields approximately 1.5 cubic feet per second throughout the year, and that the maximum freshet flow may be a hundred times this amount, or 150 cubic feet."

The location of the spillway through which the waste water flows from the reservoir must of course be determined by local conditions, but prudence demands that wherever possible it should be around rather than over the dam, even where the cost of this plan is somewhat greater. No matter where it is located, its dimensions should be selected only after careful consideration. It is now the opinion of a large number of engineers that waterworks reservoirs should not be provided with the movable flashboards on the waste weirs, which were quite common in small works until recently. These engineers claim that it is safer in the end and possibly as economical, in view of the employee's time required to watch the flashboards, to make the sill of the waste weir the maximum level of impounded water in the reservoir and never attempt to raise the water higher. This provision calls for long rather than deep waste weirs.

An old and frequently quoted rule for ascertaining the approximate length of a waste weir is to make it 3 feet long for each 100 acres in the catchment area. Just who originated this rule the writer has never been able to ascertain, but it leans toward safety, and for areas exceeding 3 square miles gives an excessive length. For smaller areas it seems to furnish a useful approximate method of calculation, while for larger areas Mr. E. Sherman Gould's formula is more in accordance with practice. The latter expression is:

$$L = 20\sqrt{A}$$

where L is the length of the weir in feet and A is the area of the catchment basin in square miles. To find the depth of water in feet, H, on the weir when the maximum flood discharge Q is passing over it, it is necessary to employ the following formula:

$$H = 0.459\sqrt[3]{Q^2 \div L^2}.$$

This height added to the elevation of the sill of the weir gives the maximum water level in the reservoir, and the dam must rise at least 3 or 4 feet above this in order to prevent it being overtopped by waves. Mr. Gould has suggested that the depth of water on the sill, when a flood volume of 150 cubic feet per second per mile is passing over a weir of length of $20\sqrt{A}$ may be expressed with sufficient accuracy for most purposes by the simple expression:

$$H = 1.77\sqrt{A}.$$

It must be distinctly understod that these formulas and rules are only fair approximations, and that after the weir is designed and the shape of the sill and ends determined, a more exact computation, using the proper weir formula for the conditions, will probably indicate a variation of perhaps as much as 10 per cent. from the first calculations. It may even be necessary to alter the length of the weir slightly to give it the requisite capacity. The latest editions of Trautwine's Pocket-Book, as well as all books on hydraulics, give such full information on weir formulas that it is unnecessary to go into the subject in this place.

The waste weir of an earthen dam is really a different kind of a dam connecting the two portions of the embankment, and it is therefore very important that the surfaces of contact where the different materials join should be so arranged as to prevent the leakage of water. In this case, again, the maxim, "water abhors

an angle," is the golden rule of success. The earth bank on each side of the weir must be held by wing walls of some form, and these walls must be carried down to good foundations. There is one not uncommon exception to this rule, which is made when the dam has to be founded on sandy soil, such as underlies all the reservoirs of the Brooklyn water-works. The method of treatment in such cases was introduced many years ago by the late James P. Kirkwood, and has been followed with success in many subsequent structures, one of the latest being the new dam of the Syracuse water-works at Skaneateles Lake. It may be best described by abridging Mr. Kirkwood's account of the construction

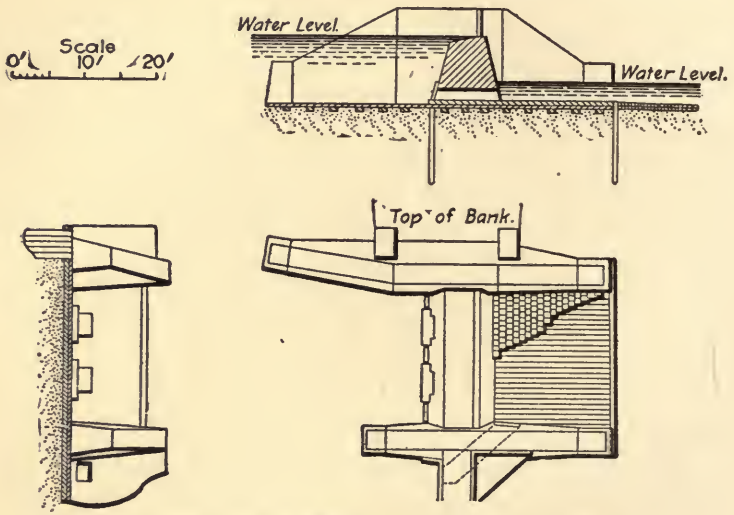


FIGURE 5.—WASTE-WEIR ON SAND FOUNDATION.

of the weir of the Jamaica reservoir dam, an earthen embankment with a puddle center wall.

The bottom of the reservoir is fine sand and gravel of the same character as the material of the surrounding plain, and the earthen dam has a puddle wall in the center made of fine gravel and sand. The general form of the waste weir, which has a clear length of 21 feet, is shown in Figure 5.

The masonry is granite laid in hydraulic cement mortar, and rests on a timber platform arranged as shown in the illustration. In order to prevent an excessive leakage of water under the plat-

form, a row of sheet pilling was driven on its upper side and extended a short distance into the bank at either end of the masonry. The specifications required this piling to be driven to a depth of at least 12 feet, but this clause was not carried out. The result was that the platform was undermined by water and had to be rebuilt with sheet piling in conformity with the requirements of the engineer. Two openings were left in the masonry, as shown in the illustration, to allow free passage of the water of the brook during the construction of the works. These openings were afterward closed and the upper face of the overfall, toward the reservoir, covered with earth. It will be noticed that the masonry of one wall has two buttresses. These were provided to break the surface and prevent leakage along the back of the wall, and as a further precaution the puddle wall was increased in width as it approached the masonry, until it covered the entire space between the buttresses. The masonry of the other wing wall was continued to form a sluiceway, which is not shown in the cut. The pavement on the apron was laid in courses with cement mortar, and was continued down-stream by a mass of rubble to protect the sheet piling at that place from eddies.

Such a method of construction would, of course, be dangerous where it was not certain that the timber platform would remain below the permanent level of the ground water.

The cross-section of the masonry of the weir is typical of the form generally given to such structures which are not much more than 10 feet high. The section is calculated by the same methods as a masonry dam. The down-stream face is usually given a slight batter, 2 inches to the foot. The design of the top is really the part calling for the most judgment, since its width must be decided upon after studying the probable character of the *debris* that will pass over it. If it is certain that no logs or other heavy masses will ever be driven against it, then its slope may be slight, say 2 inches to the foot, and its width kept down. But if logs and broken ice are to pound against it during every spring freshet, then the slope may be made a little greater, so that the logs and ice will be more likely to strike on it during floods than on the face of the dam itself. The width in this case must be ample to insure perfect security, and the stones of the sill should be large and heavy, cut to shape accurately and well bound together. Just what width to select for a given case cannot be expressed in a rule,

but must be determined by judgment aided by established precedent. The slope of the upstream face is made so as to give a dam of the requisite degree of security. If the weir is more than about 10 feet high its down-stream face should be stepped so as to break the fall of the water and diminish its erosive action on the bottom, as will be explained more fully in the chapter on masonry dams. In any case particular attention should be paid to protecting the surface on which the water strikes. This is frequently done by a pavement of stones about the size of granite paving blocks, laid in

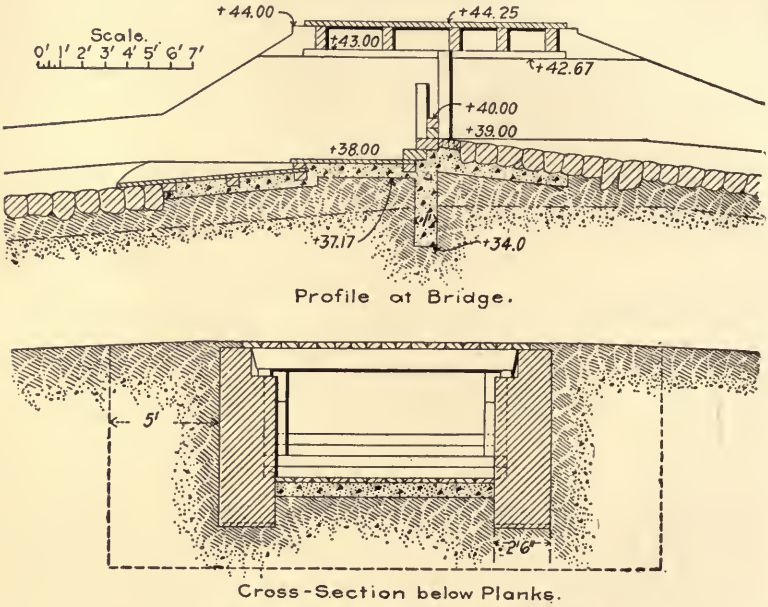


FIGURE 6.—WASTE WEIR AT NATICK, MASS.

cement mortar and resting on a well-consolidated bed of gravel or field stone, but many other methods of construction have been used.

A waste weir used on a spillway at one side of a small earth dam at Natick, Mass., is shown in Figure 6. It was designed by Mr. Desmond FitzGerald, and may be taken as a model for small work where the spillway is around and not over the dam.

Timber weirs do not possess the durability of well-built masonry structures, yet they have many good features. When con-

structed properly they last many years, and such repairs as a good timber weir requires can generally be made expeditiously and at a very small expense. The form given such weirs varies greatly, just as the form and method of construction of timber dams seem to follow no fixed types. Col. J. T. Fanning illustrates quite an elaborate weir of this sort (see Figure 7) in his "Treatise on Hydraulic Engineering." The timbers in it are stripped of bark and dressed on two sides to a thickness of 12 inches. The purpose of the sheet piling is to check the percolation of water under the dam. The timbers laid on the sills are 5 feet apart, and on these the frame is built up as shown. The sticks are united by $\frac{7}{8}$-inch

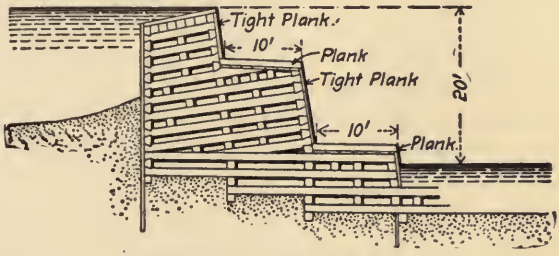


FIGURE 7.—A TIMBER WEIR.

round iron drift bolts long enough to pass through two timbers and a half way into the third. As the frame is built up, the openings must be packed tight with stone and gravel of such proportions that the work will be watertight, a matter requiring care and thoroughness to be successful. The benches and crest should be made of carefully jointed timbers laid close, and the upper and lower faces covered with tightly jointed plank. If such a weir has to be founded on rock, the bottom courses of timber should be bolted firmly to the rock.

Whether the weir be masonry or timber, a bank of gravel should be placed against its up-stream face.

CHAPTER IV.—TIMBER DAMS.

The timber dam is often regarded as a cheap makeshift, good enough for temporary use, but never to be mentioned to clients as a structure of any engineering importance and never to be recommended except for situations where other engineers will probably never see it and thus have a chance to laugh at its designer. Recently, however, engineers of high standing have designed such dams for situations where permanence and durability were necessary, and the feeling is growing that timber dams of some of the types which have stood for many years on New England mill sites and along the canals of the Eastern States are deserving of more attention. Whether or not such a structure should be built depends largely on its cost compared with earth and masonry dams, and on the nature of the work that will be necessary to replace it when complete reconstruction becomes necessary. It will sometimes be found that a small dam will impound a sufficiently large supply to enable a community to have all the water it wants for a period of 15 to 20 years, when the normal growth will require a daily supply that can only be obtained by the construction of a reservoir in another locality. In case the construction of this large reservoir should be more costly per million gallons stored than that of the smaller reservoir, then it may be good practice to build the latter, and in such a case a timber dam may afford a safe and cheap means of accomplishing this end. In a very general way it may be estimated that a large well-built timber dam will cost about one-half or three-fifths as much as a masonry dam of the same height and will last half a century with but very little outlay for repairs, while the cost may be reduced very much without impairing the safety of the structure by using less care in the construction. Small timber dams may often be constructed at a remarkably small cost. It must not be forgotten, however, that timber must be continually below the surface of the water if it is to remain in sound condition, and on this account timber dams

are more suitable for diversion weirs for directing part of the water of a river into a canal than they are for storage reservoirs.

BRUSH DAMS.

The brush and rock dam of the Western States is merely a makeshift, but in case it is necessary to build a dam not over 6 feet high across a stream having a quicksand foundation this type of a structure may serve a very useful temporary purpose. Mr. W. W. Follett, M. Am. Soc. C. E., who has an extensive acquaintance with irrigation works, recommends tying the brush, preferably willow, into fascines 6 to 8 inches in diameter. The back of the dam may be quite steep, but the front should slope very gradually in order that the water may leave the brush almost horizontally. A more elaborate affair is sometimes constructed by driving piles and building brush and rockwork around them, the whole being provided with a timber overfall, or upper covering, and apron.

In the streams where such structures are employed, the brush and rock are usually to be obtained near at hand, and when placed in position are soon silted into a fairly tight dam by the fine sand and sediment in suspension in the water. If a rapid rise carries the whole work downstream the loss is slight and can be quickly and cheaply made good again. These dams were used by the early Spanish settlers in the Southwest, and such a weir of very small size has been employed until recently, and may be still, at the head of the Zanja Madre, the main irrigating ditch of Los Angeles.

Somewhat pretentious dams of this sort were constructed near Phoenix, Ariz. According to Mr. Herbert M. Wilson, M. Am. Soc. C. E., they were built by driving stakes into the river bed across the channel. Between these, fascines of willows, about 3 inches in diameter at the butts, were laid with the butts downstream, their upper branches being loaded with boulders. Willows, cottonwood and tule reeds were again laid and covered with boulders, and this repeated until the dam reached the desired height, which rarely exceeded 5 feet. If not destroyed too soon the willows sprout and thus increase the strength of the structure, which is rendered water-tight by an up-stream filling of sand and gravel.

A dam of almost as primitive construction is used on Cherry

Creek, Colo., at the head of the Arapahoe Canal. It is 96 feet long and 10 feet in maximum height. Piles about 26 feet long and 6 feet center to center, driven in two lines across the stream, were covered on each face with 2-inch planks, so as to form a long box. About 60 feet of this box was left 3 feet lower than the rest to form an overflow weir, and the whole was then filled with sand. Adjoining this structure is a rubble masonry dam containing the headgates of the canal.

CRIB DAMS.

It is customary to draw a distinction between crib and framed dams, but the two classes frequently run together. The typical crib dam is, as its name implies, made of a number of separate cribs, usually placed in position independently and then connected by other cribs inserted between them or by bulkheads backed by loose stone. The term has been extended to embrace continuous dams framed to form cribs, but this use of the expression is somewhat confusing.

A true crib dam was described by Mr. T. C. Clarke, M. Am. Soc. C. E., in the "Transactions" of the American Society of Civil Engineers, Volume xxxiv., page 507. It was so designed as to be available for use on any kind of a foundation. If the river bed is sand, riprap is thrown on it above and below as well as at the site of the cribs, so as to protect the bottom from scouring. The cribs are 20 to 30 feet long, filled with stone, and placed in such a way that openings 5 to 8 feet long are left for the discharge of the stream during construction. The cribs are filled with stone, and form piers on which a continuous dam of triangular cross-section is built from shore to shore. It is built of 12x12-inch framed timbers protected with iron plates at the crest, and is filled with loose stone. The upstream face has a slope of 3 to 1, and the down-stream face a slope of 2 to 1. After this work is finished, the openings are closed by gates of two thicknesses of 12x12-inch timber drift-bolted together.

The crib dams built at the head of the Arizona are good examples of this type of structure, and incidentally furnish also a strong proof of the advisability of locating and building dams in torrential streams in the best manner. Both the dams to be illustrated were destroyed by floods, not so much on account of their faulty construction as on account of their unsuitability for the place and their bad location, features which the company's

engineer had forced upon him. In a stream not subject to such freshets they would probably have been satisfactory.

The design of the first weir is shown in Figure 8, and its method of construction is described in substance as follows in the thirteenth Annual Report of the U. S. Geological Survey: The bed of the stream was first prepared for receiving the dam by dumping stone on the bottom from a pontoon moored up-stream. The stone was in pieces weighing from 1 to 3 tons, and was used in such a quantity that it formed a bar across the channel over which the water passed. The stream carries considerable sediment and

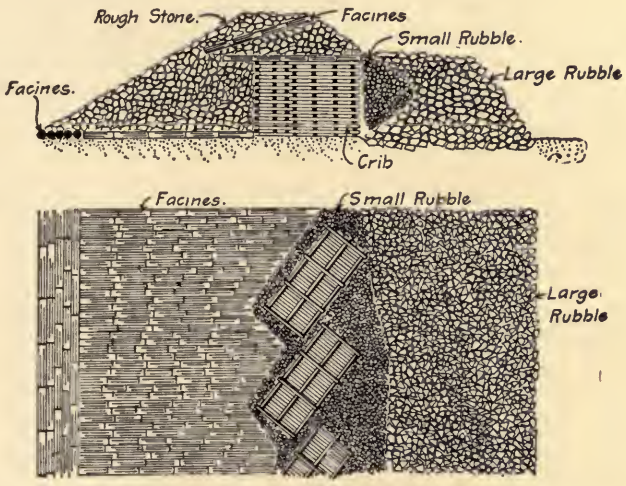


FIGURE 8.—CRIB AND RUBBLE DAM.

shingle, and the interstices in this rubble were soon filled up. The cribs were 22 feet long and 12 feet wide, and of a height corresponding to that of the weir at the place where they were to be used. They were built on shore and floated into position; when located in the diagonal manner indicated in the cut they were sunk by dumping stone on the 2-inch plank bottom spiked to the lower logs. At the up-stream toe of the wall formed in this manner a bed of willow fascines 2 feet thick was laid parallel with the direction of the current and beyond them five rows of similar fascines at right angles to the first. Boulders and gravel were then thrown on this work until the whole mass was brought to the level of the top of the cribs, where it was bound together with

fascines. The remaining features of the dam are indicated in the illustration.

The second of these weirs is shown in Figure 9, also drawn from the same source. Here the crib idea was developed in the simplest

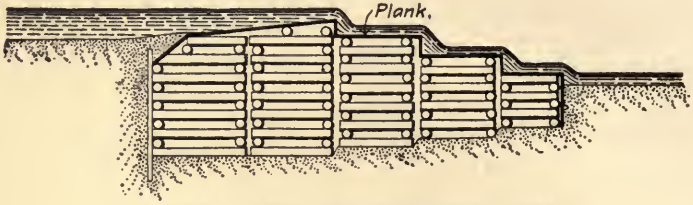


FIGURE 9.—CRIB DAM ON ARIZONA RIVER.

manner, the cross-section of the dam being formed by five 9-foot cribs of rough logs, drift-bolted and wired together, and loaded with rocks. These cribs were not constructed in precisely the same manner throughout the entire length of the dam, but the general cross-section was the same everywhere.

FRAMED DAMS.

One of the most remarkable framed dams with which the author is acquainted is that built by Mr. Robert Gilman Brown at Bodie, Cal., for the Standard Consolidated Mining Company. From a description of the structure which he prepared for the Colorado meeting of the American Institute of Mining Engineers, it appears that its leading dimensions are as follows:

Length at bottom, 80 feet; length on top, 235 feet; width at bottom, 60 feet; width on top, 15 feet; height, 42 feet; batter of water face, 1 to 1 and 1 to 2; batter of free face, 1 to 4; length of waste weir, 35 feet; depth of spillway, 5 feet.

An inspection of the details of construction shown in Figure 10 will indicate the peculiarities of this dam. In a general way it may be described as a structure of logs framed into 12-foot cribs, ballasted with earth and rock, and sheathed on its up-stream face with 3-inch plank. Only enough hewing was done to bring the logs into close contact, and the sticks were secured by drift bolts except for a few of the upper tiers, where wooden pins were used. The bottom and sides are provided with sheet piling 10 feet long, sunk into firm hardpan. Work was begun at the bottom, and after a few courses of logs were in place, the cribs were filled with

material excavated to form the higher benches. As the height increased and more material for ballast was needed, tramways were laid up and down stream on one bank at the proper elevation, and ballast was excavated from the bank and brought by cars to the dam, the tracks being shifted higher as the tiers of timber work were raised. The inclined rafters for the sheathing were carried

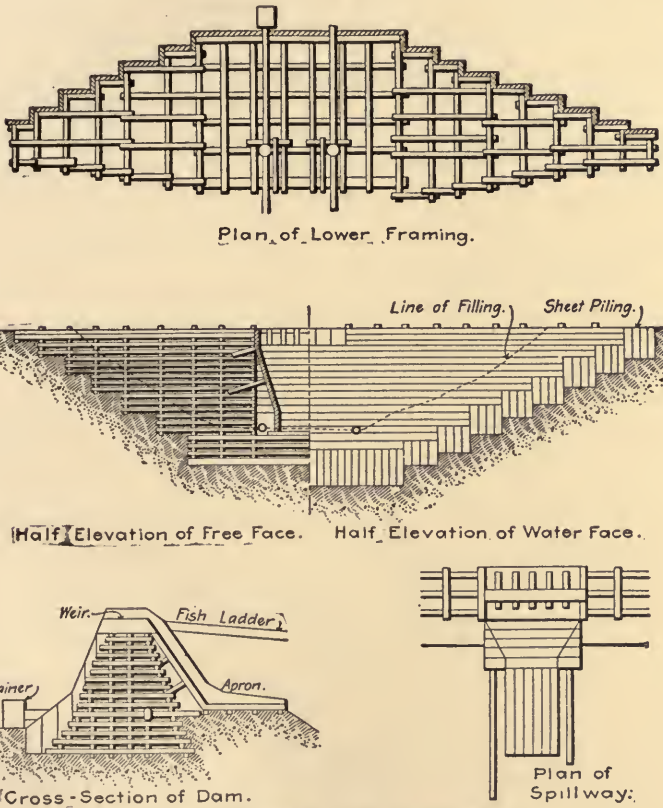


FIGURE 10.—FRAMED DAM AT BODIE, CAL.

up as rapidly as possible, and covered with horizontal planks calked carefully with tamarack bark. This sheathing was backed by clayey earth free from stones, which was tamped into place.

The waste weir is lined with planks and its sill is 28 feet above the top of the strainer. The spillway contracts from the full width of the weir at the top to 18 feet at the bottom, "with a

slight inward batter of the wings, so as better to hold the splash." The apron is made of selected straight logs with hewn joints, drift-bolted to four transverse mudsills.

The outlet and waste pipes are both apparently of the riveted type. The former commences in a wooden strainer with half-inch wire screens. Both pipes have valve chambers of plank reached by doors under the spillway. A fish ladder was provided in accordance with State laws. Mr. Brown reports that scarcely a trace of leakage has been detected below the dam, and that the valve chambers, 30 feet below the surface of the water, are dry.

About 49,000 feet, B. M., of sawn lumber were used in the dam proper, 908 logs and 472 pieces of sheet piling. The weir and spillway required 4,000 feet of sawn lumber and the fish ladder 12,000 feet. About two-thirds of the cost of the structure was the labor charge. The average number of men employed was 11 and the time required to construct the dam was 26 weeks.

Structures of this type are rarely built, since rock-fill, earth and masonry dams are generally more economical in the long run, when charges for repairs and reconstruction are taken into consideration. Nevertheless, experience in California shows that they are more substantial than is generally believed to be the case.

A much more elaborate framed dam, 13 2-3 feet high, designed to act as a weir for its whole length, is illustrated in "The Engineering Record" of April 21, 1894. This dam was built at Sewall's Falls, near Concord, N. H., and is probably typical of the best practice for such undertakings. Another structure of this class which ranks very high is that across the Missouri River at Great Falls, Mont., a description of which, by Mr. M. S. Parker, is printed in Volume xxvii. of the "Transactions" of the American Society of Civil Engineers.

Many of the timber dams constructed on irrigation works in the western part of this country are of much interest, and two of them are described here, the descriptions being based on an article on "American Irrigation Engineering" in the Thirteenth Annual Report of the United States Geological Survey. Both are weir dams, that is to say, they are built so that they may be submerged for their whole length during floods without injury.

The first structure is that across Stony Creek, at the head of the Kraft Irrigation District in California. It is 500 feet long and rests on the gravel bottom of the stream as shown in Figure

11. The wing walls at either end were formed by piles and sheathing, raised to a height believed to be sufficient to prevent the greatest flood from passing over or around them. The timber frames rest on piles and are filled with gravel; both faces are framed with 6x8-inch timbers. The up-stream face is sheathed with 3-inch planking and the lower face with 7-inch. "The foundation consists of two rows of piles driven across the entire width of the channel of the stream, 6 feet apart between centers, the two rows being 12 feet apart, one resting immediately under the foot of the timbers on the down-stream face of the weir, the other resting under the toe of an apron which extends 12 feet below the weir. Eight feet below the lower row of piles is a row of sheet piling, and 22 feet above the upper row or just above the upper

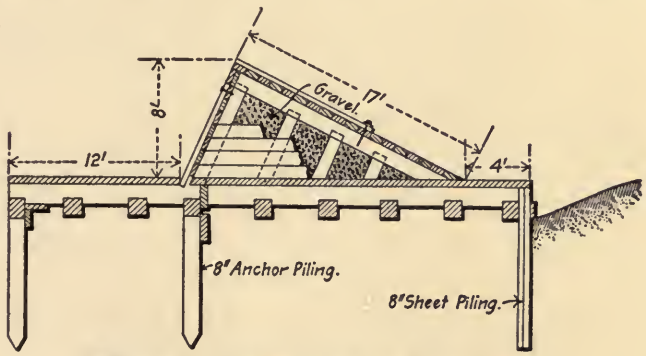


FIGURE 11.—TIMBER WEIR ON STONY CREEK.

slope of the weir is another row of sheet piling, both of these rows being of 4-inch double piling 8 feet in length and driven to bed rock."

The weir shown in Figure 12 differs from those previously described in being bolted to a rock bottom. It is built across the Bear River, near Collinston, Utah, and forms part of the headworks of the irrigating system in the northern part of the Salt Lake valley. The river at this place has rocky banks and is about 400 feet wide. The timber dam shown in the cut is 370 feet long and 17½ feet in maximum height. Its greatest width at the base is 38 feet. The greatest flood discharge of the river at this place is estimated at about 9,000 cubic feet per second, and when such a volume of water is passing the depth on the crest of the weir is

considerable. The up-stream face has a slope of 1 to 2, and the other face a slope of 2 to 1. The water falls on a wooden apron built on the prolongation of the sills of the dam. All the timbers used in the structure measure 10x12 inches. The spaces between them are filled with broken rock, and the up-stream face of the weir is protected by a mass of earth and silt deposited from the river water.

On one end of this weir is a masonry structure containing the gates of the canal on that side, while at the other end the head

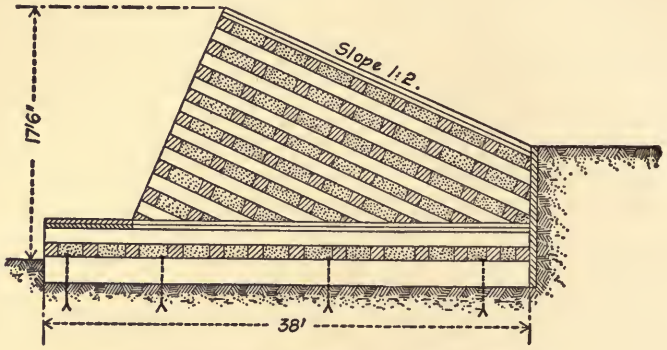


FIGURE 12.—TIMBER WEIR ON BEAR RIVER.

works of the corresponding canal are in a channel excavated in the rock bank.

In the construction of timber dams there is generally little opportunity for a choice in the wood employed. Nevertheless, it must be borne in mind that certain woods, like hemlock and green cottonwood, are particularly adapted for use under water, and a careful investigation of the varieties of available timber may have good results when an engineer has to erect one of these structures in a locality with which he is unfamiliar.

CHAPTER V.—MASONRY DAMS.

Before taking up the method of designing small masonry dams, it is necessary to call attention to a few general features of reservoir construction, which must be studied with particular care when a masonry dam is to be used. The first of these is the nature of the site. On this point, the following very general remarks by Otto Lueger (translated from his "Water Supply of Cities") give a fair idea of the influence of the site of a proposed reservoir on the design of the dam forming it:

"It is assumed that, as a rule, a clay stratum of at least 5 feet is sufficient to form a reservoir with an impervious bottom, especially in view of the deposits of silt which will inevitably be formed later and increase the tightness. In the first place, then, it is necessary to ascertain if such a deposit exists. Next, the strata below this bed are to be investigated. If the clay reaches to a great depth before encountering rock, an earth embankment is most suitable for a dam. If it is possible to satisfy the requirements with a dam 75 feet or less in height, such a structure will be safe if carefully built and the strata are horizontal or nearly so. If they are much inclined and are underlaid by material which will flow when continually moist or become so unstable that the earth above will slide under similar conditions, the construction of the dam is to be positively condemned; also in the case of nearly horizontal strata, the material below the dam must be compressed if every danger to the earth bank is to be avoided. Embankments more than 75 feet high are dangerous under all circumstances, because their saturation becomes too great under the high-water pressure. On rocky ground, embankments should not be built, but only masonry dams constructed.

"Among rocks those of eruptive stone, granite, porphyryte, basalt, trachyte, etc., offer the safest foundation for the dams. The stratified rocks do not always afford a safe bed, less on account of their strength than on account of their position and cleavage.

Nevertheless it is generally safe to build on them if they are split but little, and nearly or quite horizontal, or if the layers are inclined downward toward the reservoir from the site of the dam. Care must be taken in this case also to avoid the presence of marl or clay strata under the foundation of the dam; these might become saturated by the water in the basin and made plastic or brought to a condition in which they would flow. It is therefore

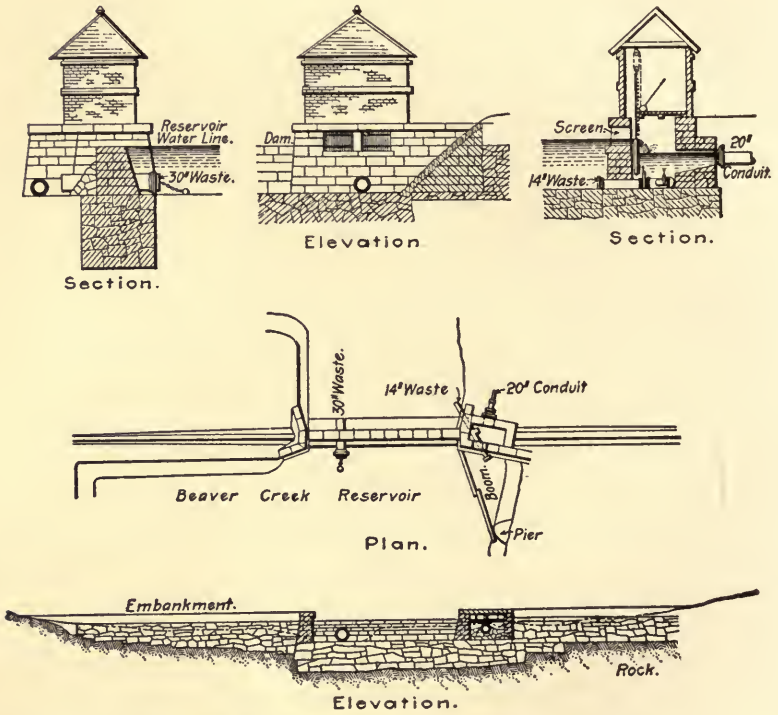


FIGURE 13.—MASONRY DAM AT LITTLE FALLS.

advisable to ascertain the character of such stratified formations by careful examinations, test borings, etc., before the position of the dam is definitely decided. This is still more important when it appears probable that cracks and deep fissures exist. If the site of the proposed dam is on the outcrop of the strata, it is possible under certain conditions that the base will be exposed to great danger of undermining, especially if a clay or marl substratum ex-

ists, as is frequently the case. Care must be taken in every case that the dam rests throughout its length on the same stratum (good dams have been built in which this rule is not followed), and that the rock bed is sound."

These remarks apply more particularly to investigations for larger reservoirs than this series of articles is designed to discuss in detail, but for the smaller works the method of investigation is essentially the same. It is necessary to ascertain if the basin will be watertight when finished, which is assured if the bottom is of impervious earth or rock without fissures leading the water under or around the natural and artificial walls of the basin.

That even the smaller works cannot always be designed with absolute certainty as regards underlying formations is shown by the experience of Mr. S. E. Babcock in building the Beaver Creek dam of the Little Falls, N. Y., Water-Works. Figure 13 and the following description of the work have been prepared from Mr. Babcock's report on this dam. The creek is dammed by a masonry structure with a spillway 50 feet wide, and forms a basin of 0.75 acre.

"The dam is 6 feet high; the south abutment of the dam is formed into a receiving chamber and inlet of conduit line, which is provided with a screen having 1-inch apertures and with a movable inlet weir, which may be raised or lowered to allow any quantity of water, from one gallon to the capacity of the conduit, to flow into the chamber and thence into the conduit; the saddle of the dam is 2 feet above the top of the conduit, and it matters not how high the water may be in Beaver Creek, the head on the conduit can never be more than a fraction over the static head of 2 feet for which it is designed. There is also a provision made for flushing the pond at the inlet chamber by means of a short length of 14-inch pipe controlled by a valve situated inside of the inlet chamber house, together with a 6-inch branch to flush out or draw off the inlet chamber. Again, the pond may be drawn entirely dry and the waters of the stream passed through a 30-inch pipe let into the center of the stone dam, closed by a plug at the upper end, provided with a deadeye, chain and ring, to which a watch tackle may be fastened and the plug removed.

"It was expected to find a water-tight foundation of rock very near the surface, upon which to start the stone dam. The surface indications on both sides of the stream showed sand-rock ledges.

Upon making the necessary excavations, the surface rock was found to be very seamy and loose and interspersed with a hard-pan formation. It was necessary to go down about 9 feet, as occasion required, to obtain a water-tight foundation, entailing an expense of over $8,000 above what was originally calculated."

One of the most surprising cases of geological conditions influencing reservoir construction is afforded by the Wentwood reservoir of the Newport, England, Water-Works, now approaching completion. In the fall of 1892 the Borough Surveyor made a number of borings on the site of the proposed reservoir, and reported the ground favorable for the construction of a suitable dam. He was appointed water engineer, and with expert assistance prepared plans for the work. A contract was made for the construction of the basin, and two years later the contractor began the excavation of the puddle trench. After going down some 100 feet, and revealing a badly fissured sandstone rock, he was ordered to suspend work. About $250,000 had been spent by this time. Mr. G. H. Hill, of Manchester, was consulted, and he reported that a geologist should be retained to assist him. Mr. Tiddemann was appointed, and an examination of the problem convinced him the reservoir could never be made tight. The resident engineer, who was a geologist as well as an engineer, was then asked to give an opinion on the matter. He advised making some practical tests of the site, and his advice was followed. Another geologist was then retained, who reported in favor of carrying out the resident engineer's tests and continuing construction if they gave satisfactory indications. By this time, so much conflicting advice had been secured, that the council retained Messrs. Hill and Hawksley to go over the whole evidence, examine the site and submit a comprehensive report on the subject. They carried out many tests, made examinations and reported in favor of totally abandoning the site. Accordingly the contract was annulled by paying the contractor a bonus of $50,000 in addition to the sum expended on account of actual construction. The resident engineer then succeeding in persuading the authorities to authorize a test of the site on a large scale. The puddle trench was filled with water, and in spite of the fissures and clefts in the rock the experiment was a highly satisfactory one. Other tests gave equally assuring indications, so Mr. Baldwin Latham was retained to examine the site. His report was so favorable that the authori-

ties voted to resume operations, conducting the work themselves, and this was done in May, 1896.

MATERIALS.

There is little to be said concerning the materials for use in a masonry dam, other than they must give a tight structure. Stone varies so widely in its character that it would be a waste of time to enter into a description of masonry here. What particular class to use in any case naturally is governed by local conditions. In all cases the masonry must unite intimately with the rock bottom, and must be carried out in such a manner that it is certain a fairly homogeneous mass of uniform strength has been constructed. Kranz says in his "Study of Reservoir Dams": "The union on all faces and beds must be irregular or cyclopean. There should be no joints in the entire structure, and pains must be taken to make the dam a single monolith. Cyclopean rubble masonry has a further advantage in that it can be easily executed to any section; the most complicated curves can be easily marked out by forms or templets, so that the masonry is easily and quickly carried ahead, a matter of no small importance."

In order to show more clearly what is meant by the term "cyclopean masonry," one much used in England, but not employed here to any extent, the work on three dams is here described, the wording of the engineer's descriptions being given in two cases, and followed very closely in the third.

Manchester Water-Works, Thirlmere dam, George Henry Hill, Engineer.—"The foundation is everywhere sunk into the solid rock, and at the river reaches a maximum depth of 50 feet below the bed. The dam is constructed of concrete, gauged 5 parts of broken stone and sand to 1 of cement; large blocks of rock from one-half to four tons in weight being imbedded in it sufficiently far apart to be properly surrounded with concrete. These blocks were not allowed to be placed within about 7 feet of the inner face. All the fine material for the concrete was produced by grinding the stone of the district. The dam is faced on both sides with heavy chisel bedded and jointed masonry of an average thickness of about 2 feet, well bonded into the concrete of the dam. The batter on the lake side is 1½ inches to a foot, and the outside is curved to a radius of 100 feet. The dam is finished at the top of the outer face by two courses of ashlar 18 inches thick,

and on the inner face by one course of the same thickness. The top of the dam carries a roadway 16 feet wide."

Vyrnwy masonry dam, George F. Deacon, Engineer.—"The mortar and concrete were put in so dry that considerable ramming was everywhere necessary to produce the jelly-like consistency which betokens incompressibility, and until that consistency was everywhere attained, ramming did not cease. The rubbing of the fine portions of lime or cement or sand into the interstices of the coarser particles during the shaking and subsequent trembling of the whole mass under the rammer cannot be attained in any other way.

"In the preparation of the rock foundation all dislocated rock was removed. Where long, steep slopes occurred in the sound rock it was benched, but with obtuse inner angles, and it was then rendered scrupulously clean with wire brushes and jets of water under pressure.

"Over the irregular surface of the foundation rock thus prepared a good coating of Portland cement mortar was brushed. Upon this the work was raised, in hollow places too small for large stones, with hand-set stones and strong mortar, and only differed from ordinary rubble-work in the greater density of its beds and joints. This greater density was secured by permitting neither masons, bricklayers nor trowels to appear upon any part of the work except the face, and by substituting for them men not too old or prejudiced to learn, with shovels, mallets and numerous ramming tools of different sizes and shapes. Whenever spaces were too small for good-size rubble-work, they were filled with mortar into which macadam size broken stone was rammed, but no previously mixed concrete was used. When a sufficiently large area was thus secured it was leveled up with mortar, among which broken stone was uniformly scattered from shovels, and beaten in with a flat beater formed of ¼-inch wrought-iron plate about 1 foot square, turned up ¾ inch at each side and having a wooden spade-like handle inserted in a wrought-iron socket riveted to the center of the upper side of the plate. The work was thus brought to a perfectly level surface on to which a bed of mortar about 2 inches thick was immediately shoveled, leveled with steel brushes, and, if it appeared unduly stiff, beaten with the same flat beater. Upon this a large stone was lowered and beaten down with many heavy two handed mallets having cylindrical timber heads 6 to 7

inches in diameter and 14 inches long. The effect was to squeeze out some mortar, even from under the largest stones, and to cause it to mount in the joints to a head of several inches. Into these joints mortar and broken stone was packed as before, and if the day's work was nearly completed the higher portions of the joints were then crammed with guano bags so completely that to the coming rain, frost, or sunshine no artificial work was exposed. It is to be particularly noted that toward the end of a day's work the joints were never allowed to be more than half-filled with mortar or concrete, but that the filling, so far as it went, was finally finished at the time by dint of firm ramming with blunt ended swords and ramming tools suitable for the various widths of joints. Thus, when the work was resumed, whether a single night or many days and nights had intervened, the junction between the new and the old mortar work had the smallest possible area, and had been thoroughly protected from the weather in the interval."

In order to reduce the hydrostatic pressure on the base, due to the percolation of water through the seams of the rock, along the base of each of the more important beds of rock, not within 15 feet of the face of the dam, a drain was formed in the masonry between 6 and 9 inches square, and from these drains funnels were carried up in different vertical transverse planes of the dam to above the backwater level. The funnels all issue at the side of a longitudinal tunnel 4¼ feet by 2½ feet, so that the flow from each is rendered visible. From this tunnel a cross tunnel serves as an outlet for the water from the rock and as a passage to the main tunnel.

Boyd's Corner dam, J. J. R. Croes, M. Am. Soc. C. E., Engineer in Charge.—The lower portion of the Boyd's Corner dam of the New York Water-Works consists of concrete in which large unworked stones were laid, while in the upper portion concrete alone was used. Above the bed-rock the concrete is faced on both sides with coursed stone having cut beds and joints. The mortar used was composed of one part of hydraulic cement to two parts of sand, by measure; the proportion of mortar to stone was such as to fill all void spaces, and be in excess of the latter not more than 10 per cent. The concrete was laid in courses of 6 inches and well rammed. The large, rough stones were laid in full beds of mortar, and their surfaces covered with mortar half an inch thick immediately before laying concrete around them.

In a description of this work written by Mr. Croes, it is said that from the beginning to the end of the construction there was a continuous struggle on the part of the engineers to have the sand for the mortar properly cleaned and screened, and of the contractor to avoid it. Several methods of washing were tried, of which the most successful consisted in spreading the sand in a layer of about 3 inches depth on the bottom of a shallow box, 6x12 feet, slightly inclined, and playing on it with a hose from a force pump. In freezing weather the mortar was mixed with salt water, the rule for the preparation of the brine being to dissolve 1 pound of rock salt in 18 gallons of water when the temperature was at 32° Fahr., and add 3 ounces of salt for 3 degrees lower temperature. The masonry laid with mortar thus prepared stood well, and showed no signs of having been affected by the frost. The experience gained on this work indicates that it does not pay to use large blocks of stone, when the thickness of the dam is less than about 10 feet.

The Sodom Dam, Alphonse Fteley, Chief Engineer.—The Sodom dam of the Croton Water-Works is regarded as a remarkably tight structure, and for this reason the following notes concerning its construction have been condensed from an article by Mr. Walter McCulloh, M. Am. Soc. C. E., who was connected with the work from its commencement.

The bed of the river was prepared by blasting with light charges of 40 and 60 per cent. dynamite until firm rock was reached. All loose seams were followed up with block holes and black powder blasting and by barring out until a solid and practically tight bottom was secured. The surface prepared in this manner was swept with wire stable brooms and washed clean with streams from hose pipes.

When the bottom was ready, it was decided to cover it with rich Portland cement concrete so as to form a series of small level beds on which to start the rubble masonry. This plan was abandoned after a two day's trial, because it was found a better bed could be made with a rubble of small stones. "A large quantity of water made its way through the loose rock above the bottom, and in many places through seams in the bottom itself; but in these cases, where the rock was solid, the seams were not followed any deeper. The springs in the bottom would wash the mortar out of the concrete, and in many cases render it worthless; but in making the

rubble beds, the water could be led round and prevented from doing harm. These streams were nursed about from place to place till finally a small well, 2 feet in diameter and 1 to 2 feet deep, would be formed just around the point where the water boiled up. When the mortar about each little well had thoroughly set, the water was bailed out, the well quickly filled with dry mortar, a bed of stiff, wet mortar put on top of this, and on top of all a large rubble-stone was placed." It will be noticed that the principle governing this treatment of ground water is the exact opposite of that of the method of Mr. Deacon on the Vyrnwy dam, which is also that of some French engineers; that is to say, the construction of drains through the masonry to take away the water from the springs.

The rubble masonry laid below the bottom of the river was in a 2 to 1 Portland cement mortar. Above that level, on the up-stream face, it was given a protection of facing-stone 30 inches deep, and also above the level of the ground on the down-stream face. The rubble backing of the central portion of the wall was laid in a 3 to 1 Portland cement mortar. Rubble-stones varied from a cubic foot to a cubic yard in bulk, and in placing them the beds of mortar were made very full, and the stone thoroughly shaken to a firm position. The rubble was not carried on in level courses, but was broken as much horizontally as possible, so as to avoid having a straight joint of mortar through the wall. In filling the interstices the rule invariably followed was to put the mortar in first, then force into it all the spalls it would take, thus insuring perfectly full joints and as much stone in the work as possible. Grouting was not permitted at all. All stones of whatever size were thoroughly washed before going on the wall, and were usually wet when placed in the work. The rubble-stone was a hard and tight-grained gneiss of irregular cleavage, obtained about 1¼ miles from the dam, while the facing-stones were a light limestone obtained 7 miles distant at a quarry opened for the dam expressly.

Southington, Conn., Dam, T. H. McKenzie, Engineer.—The dam of the distributing reservoir at Southington is an example of masonry structures not resting on rock. "The bed of the stream was a quicksand, which lay very firmly in its natural bed. The foundation was prepared by excavating two trenches parallel with the face of the dam to a depth of about 3 feet. Sills were laid at

the bottom and top of the excavation and sheet piling driven down and spiked to the sills. The trenches were then filled with concrete, and a layer of concrete 1 foot thick by 15 feet wide covers the entire surface under the dam. The dam is built of granite rubble masonry. The stone was quarried within 1,000

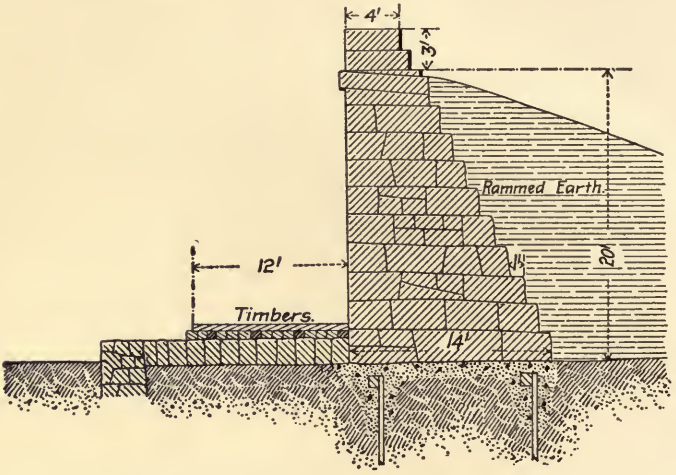


FIGURE 14.—MASONRY DAM AT SOUTHINGTON.

feet of the dam. Every stone was cleaned and wet before laying, and was laid in a full bed of cement-mortar; all interstices were filled with mortar and stone driven into it." Figure 14 is a cross-section of the dam through the overflow.

EARTH BACKING.

It will be noticed that the Southington dam has an earth backing, a feature approved by some engineers and condemned by others. One of the most striking examples of such construction is the Dunnings dam at Scranton, Pa., built under the direction of Mr. E. Sherman Gould, M. Am. Soc. C. E. This is a very interesting structure in many respects, particularly as the masonry dam proper rests for part of its length on rock and for the remaining distance on "fine sand and gravel, quicksand and cobblestones —hard, compact, inelastic—a natural concrete in fact," into which a bar could not be forced more than a foot or so by working

or driving. The whole of this masonry dam is backed by earth, and Mr. Gould gives the following explanation of his reason for this backing:

"Had the inside embankment been confined to the northerly portion, a very heavy and expensive retaining wall projecting into the reservoir would have been needed, costing more than making the bank continuous. A second reason was the conviction that such a backing is always an advantage, even to a masonry dam. It impedes leakage and is equivalent to a deepening of the foundation on the water side. A dam built in this way may be regarded as an earthen dam in which the exterior slope of earth has been replaced by an equivalent mass of masonry applied to the front of the center wall. This is the form of dam of medium height which the writer would always recommend when a rock bottom can be found. If the foundation must be upon earth he would hesitate to adopt it, for he would consider the extra concentration of weight on a smaller surface at the junction of the two different materials, masonry and earth, as disadvantageous, and, besides, would probably feel that the more extended earthen bank would be needed to smother down any percolations from under the center wall which might otherwise show in front of the dam."

Other engineers hold an exactly opposite opinion. Mr. James D. Schuyler, M. Am. Soc. C. E., in describing the remarkable Sweetwater dam, built by him, states: "The (proposed) combination of earth and masonry was rejected, as it seemed to the writer that water was sufficiently heavy for the masonry wall to support without adding the last straw on the camel's back, of a mass of saturated earth."

Such divergent opinions cannot be reconciled easily, yet the writer believes that both views are largely correct, basing his opinion on the following reasoning: Mr. Desmond FitzGerald, M. Am. Soc. C. E., has shown conclusively by some careful experiments on one of the best earth dams ever built, that the earth of such a dam on the upstream side of a tight masonry core-wall is well saturated with water. His experiments were conducted by means of a series of open vertical tubes sunk in a line across the dam. Water rose in the tubes on the upstream side of the core nearly to the level of that in the reservoir, while on the other side of the core the earth was practically dry. The fact that the earth was so saturated, however, does not necessarily prove that the dam

would leak if the masonry were replaced by good puddle, for many all-earth dams are as tight as any engineer could wish. The water may be in part of the earth, yet encounter such enormous frictional resistance in passing through the almost microscopical interstices in a well-built bank that the head which tends to force it along is completely frittered away. Consequently it is believed that where a comparatively low masonry dam is built on a rock bottom, the use or disuse of an earth fill is a matter of local conditions. If it is cheaper to use one than to build retaining walls, as in the case of the Dunnings dam, the writer would do so, proportioning the masonry to carry some earth pressure, as Mr. Fitz-Gerald's experiments indicate such a pressure probably would exist. If the earth backing would add expense to the structure, he certainly would not use it, as he believes a sufficiently safe and impervious dam can be built of masonry alone.

The Southington dam, however, was not founded on rock but on a firm quicksand, and it will be noticed that this material was disturbed very little in construction. Why an earth dam was not used at this place the writer is unable to state, although he has no doubt that the reason was simply that the masonry structure was the cheaper. As a matter of fact, many low masonry dams have been built on earth foundations, and have given good satisfaction, particularly on Indian irrigation works. A backing of earth on such a dam is not necessary in the writer's opinion, but it has two advantages. In the first place, it adds just so much material for the water to pass through before it can go under the dam. The earth at the dam has been more or less disturbed during construction, and the fill above it increases the frictional resistance to percolation and consequently the liability to leakage at the weakest part of the bottom. In the second place the earth fill probably tends to produce a more satisfactory distribution of pressure on the earth bottom, a matter of no consequence with a rock bed; the mathematical investigation of this feature involves so many assumptions as to be of little value, but the writer considers that the weight of backing against a masonry dam must tend, under certain condition, to make the distribution of pressure on the earth about the base of the masonry more nearly uniform than is the case where there is no backing. With good hardpan such an argument would be of little importance, while with some other classes of earth foundations, on which circumstances might com-

pel the construction of a masonry dam, this matter would probably be worth consideration.

DESIGN.

The design of a low masonry dam is a matter calling for little of the skill which the preparation of the plans of a high dam demands. Experience has practically settled on the form shown on the left in Figure 14a as that generally useful for structures up to 15 feet high, not liable to be overflowed, and built in places where suitable stone is cheap. The dam should not be much narrower on top than is shown in the cut, unless it is short and the reservoir covers a small area. It frequently happens that the upstream face

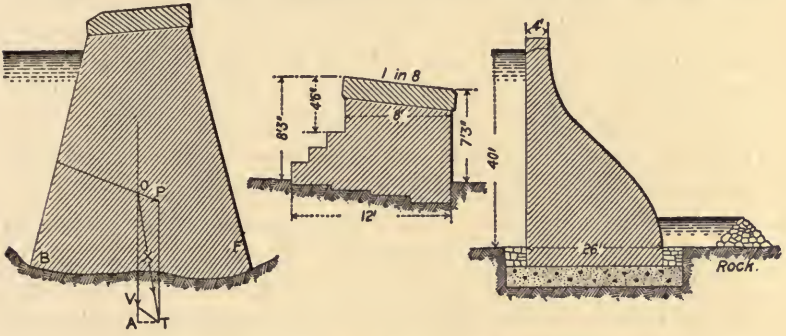


FIGURE 14a.—TYPES OF SMALL MASONRY DAMS.

of the dam is more nearly vertical than the other, as is the case of the middle structure shown in Figure 14a.

The principles governing the stability of such a structure may be best indicated by examining the stability of the dam shown on the left in Figure 14a, which is a New Jersey structure.

In the first place it must be able to withstand any forces which tend to slide any part of the masonry over that below the level at which the tendency to slide is assumed to exist. In any dam of the class under construction such a danger can be made so remote by avoiding all through joints in the masonry and setting the stones in good hydraulic mortar that it is unnecessary to investigate this feature farther.

In the second place, the greatest pressure per square inch on the masonry dam at any point of the dam must not exceed a certain amount, usually assumed to be about 150 pounds. This feature is the governing one in high dams, and has attracted the attention

of some of the ablest theorists among engineers; the literature on the subject, especially in French, is of much interest, but does not need discussion in this connection, other than that given in a following paragraph.

In the third place, the dam must be amply strong enough to resist any tendency to overturn at the base or any higher section. Through the center of gravity of the portion of the dam above the given horizontal section, which is assumed to be the base in this case, let fall a vertical line, and from a point two-thirds of the distance from the high-water level to the given horizontal section, draw a line perpendicular to the face of the dam. Rules for finding the center of gravity of a plane figure are given in Trautwine's "Handbook" and similar works. These two lines will intersect at some point, O. On the vertical line lay off OV equal to the weight in pounds of so much of a vertical slice of the dam 1 foot thick as lies above the section, and on the second line lay off OP equal to the hydrostatic pressure on the part of the dam under consideration. This pressure is 31.2 times the square of the depth of water in feet. These two distances must be laid off to the same scale. The two lines are the sides of a parallelogram, of which the diagonal OT gives the direction of the resultant force acting to overturn the dam, the point X at which the line of action of the force cuts the base of the section, and the magnitude of the force. The last is found by measuring the diagonal with the same scale used in laying off OP and OV. Call half the width of the horizontal section a, and let the distance from X to the middle point of the section be b; the factor of safety of the wall is equal to a divided by b. The wall is not secure unless this factor of safety is at least three, but in most masonry dams of low height it is generally so much more that a mere glance at the section will convince an engineer of its security. In the figure, the factor is about eleven. The total pressure to which the horizontal section is subjected may be found by drawing TA perpendicular to OV and measuring OA with the same scale used in laying off OP and O V. Since the point X is not at the middle of the horizontal section, the pressure on the section will not be uniform, but will be greater at F than at B. The amount of this maximum pressure may be found by means of the following equation:

$$C = \frac{2D}{b}\left(2 - 3\frac{t}{b}\right)$$

In this equation C is the maximum pressure in pounds per square foot, D is the total pressure in pounds on the base, as found by measuring the line OA, b is the width in feet of the section BF, and t is the distance XF in feet. This method of finding the maximum pressure must only be used when FX is greater than one-third BF and less than two-thirds BF. If this condition is not fulfilled something is wrong with the design and it should be changed.

A study of this graphical process of ascertaining the stresses of a dam will show clearly the effect of changing the cross-section of the dam. If the back of the wall on which the water presses were vertical, the center of gravity of the masonry would be thrown toward B, while if the dam were made thinner at the same time the ratio of FX to FB could be maintained constant. It might seem a good plan to do this, but a moment's reflection will show that a dam has other forces to resist beside hydrostatic pressure; it must be able to withstand the pressure of ice and possibly the passage of ice and drift over its crest in floods. Mathematics offer slight assistance in estimating the stresses such forces exert on a low dam, and experience is the best teacher in such matters. In the case of the dam shown on the left in Figure 14a, it is unnecessary to investigate the security of the structure when the reservoir is empty, but in most cases this should be done.

The middle cut in Figure 14a shows the cross-section of a low masonry dam designed by Mr. John W. Hill, M. Am. Soc. C. E., for the Findlay, O., water-works. This structure is made very heavy to withstand the floods which are liable to submerge it 10 or more feet, and affords an example of a structure of about the maximum security, too heavy indeed for ordinary purposes. It is of interest in this connection, however, as illustrating the use of steps on the down-stream face of a dam. These steps break the force of the water passing over the crest of the dam and prevent the wearing away of the bedrock on which the water strikes. Unless this bedrock is an exceptionally hard stone it is best to break the fall of the water by steps every few feet, or else give the face of the dam a reversed curve such that the water will tend to leave the masonry of the structure at only a small angle with the river bed. Aprons of timber or stone blocks are generally used below a dam to protect the bedrock, although they are sometimes replaced, as shown

in the section on the right in Figure 14a, by a water cushion, or pool of water formed by a small secondary dam a short distance below the main structure.

This dam was built by Mr. J. W. Ledoux, M. Am. Soc. C. E., for the water-works of Greenville, S. C., and may be taken as typical of small curved dams. The dam forms a reservoir in which is impounded the water from a catchment area of about 1 square mile of densely wooded mountainous country. The masonry is granite rubble laid in a mortar composed of one part of hydraulic cement to two parts of sand. The stone was taken from a ledge 400 feet upstream and hauled on a tramway from two derricks at the quarry to two on the dam. The cement cost $1.64 on the cars and was hauled 3½ miles in carts, and the sand 1 mile over ordinary country roads. Each cubic yard of mansonry required 1.4 barrels of cement. The masons were paid $3 on an average and common laborers $1 a day. The masonry cost $6.50 a cubic yard exclusive of the coping.

SPECIFICATIONS.

Something should be said concerning the specifications for stone dams, as there is a tendency to make them very elaborate even when the structure is a small one. It is now pretty well recognized that specifications for such dams must be general and refer the contractor to the engineer for his detailed information, rather than be minutely detailed. In the matter of cement, for example, some of the most important recent work of this class has been let under specifications which are without any of the exact requirements considered necessary in masonry work for some other purposes. Mr. A. Fteley, for example, wrote the following specification for the Titicus dam of the Croton water-works, and practically the same wording is used in a recent Boston specification:

"American cement and Portland cement are to be used. The American cement must be in good condition and must be equal in quality to the best Rosendale cement. It must be made by manufacturers of established reputation, must be fresh and very fine ground, and in well-made casks (or equally safe and tight receptacles approved by the Engineer). The Portland cement must be of a brand equal in quality to the best English Portland cement. To insure its good quality, all the cement furnished by the contractors will be subject to inspection and rigorous tests; and if

found of improper quality, will be branded, and must be immediately removed from the work, the character of the tests to be determined by the Engineer."

These requirements throw the whole responsibility of the character of the cement on the engineer, and enable him to reject poor cements which he might be forced to accept if this material were supplied under the more elaborate requirements in favor until recently. Elaborate cement specifications are of value in certain works, but the prevailing opinion is that they are not adapted for securing the most satisfactory construction of dams.

The rubble backing of a small masonry dam at Skaneateles Lake was built according to the following specifications:

"The backing stones of the dam.....shall be of sound, well-shaped and durable stones, and in general not less than 6 inches in thickness, nor less than 3 feet area of bed. The edges of all thin wedge-shaped stones, before they are placed in the work, must be broken off to a thickness of 6 inches or more. The hearting stones shall be laid on their broadest beds, and laid so that no two vertical joints shall be opposite, or in a position to form a straight line through the wall. No pinning, wedging or leveling up with spalls that shall raise the stones from their beds will be permitted. No joints or spaces between stones will be permitted to be filled with spalls until after the mortar is in and completely fills the space; then thin pieces of stones or spalls will be gently driven in. All the masonry shall be laid in full beds of mortar and the joints shall be flushed full to the top as soon as the stone is placed. No grouting will be permitted. No moving, dressing or hammering of the stone on the wall, that will disturb the setting of the cement, will be allowed."

The facing stones of the dam have to be provided for more definitely. In the case of the masonry dam of Reservoir No. 5 of the Boston Water-Works, now a part of the Metropolitan system, the specification reads:

"The outer faces of the masonry dam.....are to be made of range stones.....of unobjectionable quality, sound and durable, free from all seams and other defects, and of such kind as shall be approved by the Engineer. All beds, builds, and joints are to be cut true to a depth of not more than 4 inches and not less than 3 inches from the faces, and to surfaces allowing of one-half inch joints at most; the joints for the remaining

part of the stones not to exceed 2 inches in thickness at any point."

This dam is a fairly large structure, and the specifications for face stones for it correspond with those for similar works on the Croton water system of New York. It should be stated, however, that many engineers prefer more stringent requirements for facing masonry, and an example is given in the following extract from the specifications for a dam at Skaneateles Lake, already quoted:

"The face of the masonry in the dam is to be of a heavy class of broken range ashlar, having horizontal beds and vertical joints, to be dressed to form a mortar joint not exceeding one-half inch in thickness for a depth back from the face of the wall of not less than 10 inches on the beds, nor less than 6 inches at the end joints. The stones for the face of the wall must generally be not less than 10 inches thick, nor less than 2½ feet long in line of the wall, nor of a less width of bed for stretchers than 12 inches, and in no case less than 1¼ times the depth of the stone. The bond of the face stones in general shall be not less than 10 inches and in no case less than 9 inches with each other, or 9 inches with the backing. In forming the bond between adjoining face stones of varying thickness, properly prepared levelers not less than 4 inches in thickness may be used, but where a difference of level of less than 4 inches occurs, the bonds shall be made by cutting a check in the stone. Headers occupying at least one-fifth of the face of the wall, to be not less than 18 inches wide or 10 inches in depth, on the face, and in general not less than 3 nor more than 3½ feet long, shall be placed in front and rear of the wall, so that those in the rear shall be intermediate to those in front. All face stone shall be laid on their quarry beds."

The specifications for masonry dams should also contain the usual clauses for insuring good materials and workmanship, which it is unnecessary to outline here. The best drawn specifications will not insure a good structure unless the work is done under the constant supervision of a competent man with sufficient authority to prevent any poor work being done. Thorough inspection is the secret of good work of this class, and such inspection cannot be done properly by anyone who is not perfectly familiar with the laying of first-class masonry. The writer is strongly of the opinion that masonry dams, small or large, should never be built

except under the constant supervision of such a man. The so-called inspection performed by instrumentmen or draftsmen not needed elsewhere will be of little value.

ROCK-FILL DAMS.

A type of dam which has been used extensively in the West is made of loose rock and has received the name of rock-fill. Its origin was probably a loosely built timber dam in which the timbering manifestly added little if anything to the security of the structure. It was but a step from such a dam to one built wholly of loose rock, and the iconoclastic engineers who constructed the earlier works in the new States and Territories evidently found it an easy one to take. The fact that some missteps were taken at the same time should not be considered as totally condemning their whole course in the matter. Sometimes they combined the crib and rock-fill constructions in one dam, like the Bowman dam in California. In this case a crib dam was built first and then the height raised to twice the original level by means of carefully placed loose stones on the down-stream side and on top of the timber work. The structure is about 100 feet high and is faced with plank like most of its type.

Mining dams have often been built without any cribwork whatever. The Fordyce dam in California, for example, is about 70 feet high with an upstream batter of 1 on 1 and a downstream batter of 4 on 1. It is 90 feet wide at the bottom, 6 feet on top, and consists of an interior mass of loose stones with carefully placed stones on each face, the upstream face being rendered tight by 3-inch plank. In a few cases successful attempts have been made to render such dams water-tight by placing an earth bank on their upstream face; the dams on the Pecos and Boise irrigation systems are examples of such construction.

In all these structures the purpose of the rock is simply to give weight, the tightness of the dam depending on the facing. It is evident that if the facing gives way when the reservoir is full, there is a strong probability that the stones will be washed away and the valley below flooded. This has happened several times, and it is therefore advisable to construct such rock-fill dams only at such sites as render it certain no harm will be done farther down the valley in case the structures fail. In their proper place, however, these dams give good satisfaction.

Rock-fill dams formerly cost from $2 to $3 per cubic yard, but modern machinery and methods have reduced these figures very much. Mr. R. B. Stanton, M. Am. Soc. C. E., has given the following data as to the cost per cubic yard of the items of a dam of this type. Quarrying rock, 6 cents; loading buckets by hand, including breaking large rocks with powder, 20 cents; hoisting and conveying rock, 6 cents; placing rock on dam, 3 cents; cost of plant, 10 cents; total cost, 45 cents; common labor was paid $1.75 a day and coal cost $10 a ton. The plant consisted of one Lidgerwood cableway with three derricks on the dam for distributing and placing rock. Quarrying was done by exploding several tons of powder in drifts and shafts, thus breaking up from 25,000 to 30,000 cubic yards of rock in one shot. The total amount of rock in the dam was about 120,000 cubic yards.

CHAPTER VI.—SPECIAL FEATURES OF RIVER AND POND SUPPLIES.

Irrigation enterprises have furnished many valuable precedents for the construction of water-works for domestic supplies, and among these are various provisions for removing sand and sediment from the water of the streams. Where a valley is crossed by a dam forming a basin of such size that the water within it has practically no current, the tendency is for all the suspended material in the water to sink to the bottom. In such a case, no provision has to be made for the removal of sand from the water, since it has probably been deposited long before the water reaches the intake. In other cases, the stream is large and its current so slow that little sand is held by the water, and it is only necessary to lay a suction main out into the river and pump the water needed. In still other cases, the stream is a swift mountain river, full of sand and small gravel, especially in times of flood. If this material is admitted to the conduit, it will erode it rapidly and cause much damage at the gates and other specials, besides tending to clog the pipes at every low point on the profile. It frequently happens the river is so large that only a low weir is needed to ensure an ample supply for the conduit at all times, and the surplus flows over the weir along the old course of the river. It is usual to have a spillway on these weirs, but they are often inadequate to pass the full flood volumes.

The conditions governing the design of headworks to intercept the water of a sediment-bearing stream have been stated very fully by Mr. William Ham. Hall, M. Am. Soc. C. E., as follows: "(1) to interpose, in the form of a dam across the river canyon as little obstruction to the free flow of its high floods as possible; (2) to keep the ordinary flood and low-water flow of the stream permanently in a channel next to the intake: (3) to prevent the lodgment and accumulation of detritus at or immediately above or below the intake; (4) to draw the clearest available waters into

the canal heading; (5) to sluice the heavier detritus and more heavily laden waters rapidly by the same." In order to fulfill these conditions Mr. Hall advises the adoption of two features in the plans wherever practicable. The first of these is taking water for the main conduit from the surface of the natural stream, in a thin sheet over the lip of a long weir, and with the least possible deflection of the line of flow—that is to say, by making the weir as nearly parallel as possible to the direction of the current. The second feature is flushing away the waste water at the bottom, not top, of the waste weir and giving it a high velocity by sloping the channel leading to the sluice gates. "An underflow gate draws from the bottom of the stream and is therefore the proper design for a sluiceway to get rid of the sands and gravels carried near or rolled along the bottom. It should not be used for an intake, for, thus used, it directly defeats one of the primary objects desired. Overflow gates, usually put in as flash-boards, can be designed so as to admit of handling with perfect ease under any circumstances. They draw from the top of the stream first, and heavier detritus has to accumulate before passing over them. They are, for this reason, detrimental in a by-pass sluiceway, but exactly applicable for intake gateways; and, where the waters are carrying sediments, the intake should be in a thin sheet over such gates, so as to draw only from the surface of the natural stream."

Another method of accomplishing the same object has been described by Mr. L. L. Tribus, M. Am. Soc. C. E. The plan was to take water from the river in a sheet 2 inches thick and about 100 feet long. The water passed in this way into a chamber from which it flowed under a suspended apron into another chamber, where it rose and finally escaped over a second weir. This plan provided for the withdrawal of the clearest water in the stream, and its circuitous flow at a very low velocity to the beginning of the conduit, conditions favorable to the further clarification of the supply by sedimentation. Theoretically the transporting power of a stream varies as the sixth power of its velocity, and although the theoretical conditions are never realized, it is fortunately true, speaking generally, that sedimentation proceeds more rapidly than the simple decrease in velocity of a current. Advantage is taken of this fact in the construction of sand pits along the line of open conduits, as will be pointed out later, and Eng-

lish engineers make use of the same principle in designing what they call residuum lodges, small tanks or basins at the upper ends of reservoirs in which silt or other sediment is removed from the water of feeders before it enters the reservoirs proper.

The use of lakes and ponds as sources of supply requires more skill in the original design and subsequent operation of water-works than would at first seem necessary. The reason for this is to be found in the phenomena caused by the changes in temperature of the water, which have been studied carefully by Messrs. F. P. Stearns and Desmond FitzGerald and should be understood by the manager of every water system.

In ponds less than about 25 feet in depth there is not much more than 5 or 6 degrees difference in the temperature of the top and bottom layers of water, but in deeper ponds, where the wind cannot stir up the whole of the water, a different condition exists. To understand what takes place it must be recalled that water is densest, or weighs most per cubic foot, when its temperature is 39.2 degrees. In the winter the natural tendency is to have the bottom layers in a deep pond of about this temperature, and the ascending layers more and more cold until the water just below the ice is practically at the freezing point. This condition remains until the ice melts in the spring, when the surface water is warmed and consequently becomes more dense. Under the influence of the increasing warmth and winds the layers change their positions until finally the whole volume of water has a practically uniform temperature and is consequently in a condition of unstable equilibrium.

As the season progresses, however, the top layers of the lake are heated a few degrees above those below, and another state of stable equilibrium ensues, with the coldest water at the bottom, the contrary of the case in winter. During the summer, the layers more than about 15 feet below the surface are stagnant and unaffected by the wind. As autumn passes, the surface water cools and a reversal of the conditions in the spring occurs, resulting in another state of unstable equilibrium, when the entire contents of the pond or lake are turned over.

The effect of these phenomena has been stated generally by Mr. FitzGerald in the following words: "In a lake with any considerable amount of organic matter in it and also in deep artificial storage reservoirs, where the surface has not been stripped, the

lower layers, which are quiescent during the great stagnation period, gradually collect all the organic matter from the upper layers, and decay goes on until the oxygen is used up. The water becomes darker and darker, until by October it is very yellow and generally has a disagreeable smell. Of course, when the great overturning comes, in November, all this bad water is brought to the surface, and the infusoria and diatoms begin to grow in enormous numbers, because the organic matter and oxygen are brought together and provide food for organic life. The same phenomenon takes place in the spring period of circulation, although on a smaller scale." Two important practical lessons which these phenomena teach are the following:

1. In drawing water for use from a deep pond or reservoir during the two periods of stagnation, it is desirable to take water from near the surface, and if there is a surplus in the basin, to waste the bottom layers, which contain the most impurities, so that it will not be mixed with the better water during the next overturning.

2. "Many engineers are disposed to sneer at the idea of the necessity for removing all the organic matter from the bottom and sides of the valley which is to form a storage basin for a domestic supply. There is a marked difference in the condition of the water below the 20-foot line in the summer in a properly prepared basin and one that is not treated. In the basins on the Boston Water-Works which have been stripped of loam, stumps, etc., and have had their shallow flowage removed, the water is comparatively good all the way to the bottom, even in October, when the effects of a long period of stagnation are best studied. Oxygen is present, showing that there is not enough organic matter present in a state of decomposition to use up the oxygen, the organisms are few, because there is not sufficient food to support large growths, and the amorphous matter is small in amount. In a sheet of water not so treated, however, we find a very different condition of affairs. There is no oxygen at the bottom, a high color, much organic matter (where decay has been arrested from a lack of oxygen), and a considerable amount of amorphous matter. All of these objectionable characteristics are distributed throughout the whole vertical section on the overturning, in November, resulting in large growths of diatoms and infusoria. It is no wonder that the water occasionally tastes bad under these

circumstances." (Extract from a report by Mr. FitzGerald, dated January 1, 1895.)

As this is an important matter to the owners of every water-works drawing its supply from a pond or lake of any size, a brief résumé will be given of the results of Mr. Stearns' investigations of another phase of this subject, which were made with much care. It is of course necessary for all organisms to have food, which, in the case of the low forms of plant and animal life in ponds and lakes comes either from sewage and manures or from the vegetable matter in the bottom of the basin. Even if sewage is turned into a cesspool and filters a long distance through the ground, it will still contain a large amount of food material for the organisms, and may have nearly the same effect in promoting their growth as it would had it been turned directly into the water. The second source from which this food may be derived, the vegetable matter at the bottom of the pond, is shown in the case of the Ludlow Reservoir, near Springfield, Mass. The amount of food material in the water in summer, when the growth of algæ was greatest, was three times that of the winter. The water in the feeders entering the reservoir contains little of this material, which points to the bottom as the source of the supply. "With regard to the depth and size and absence of very shallow flowage, this reservoir ranks high among those of the State. As a further indication that depth is less important than the food supply, the case of Pilling's Pond, in Lynnfield, may be cited. This is a very old storage reservoir, made for mill purposes by flowing a large level meadow to a depth of 4 feet. The average depth of the pond, including the shallow portions near the edges, is about 3 feet. At the time of the examination it was kept constantly full. The area of the pond is in the neighborhood of 85 acres. Examinations made during the summer of 1889 showed that, notwithstanding the small depth and the consequent high temperature of the water, which at times reached 80° Fahr., the water did not contain any abnormal growth of organisms nor become offensive. This comparative favorable result appears to be due to the fact that the reservoir is so old that the available food supply has been removed from the bottom."

In this connection it may be added that a number of experienced engineers have reported in favor of a thick gravel filling on top of the muck or peat at the bottom of proposed reservoirs,

where the cost of removing the vegetable matter was too great to be met by the resources of the builders of the works. For the same reason no pockets should be left in the bottom of reservoirs at places where there is any liability of the water becoming shallow. Such depressions should be filled with clean material in order to prevent organisms flourishing in them during dry seasons.

The worst odors in drinking waters are due to floating microscopic organisms, which are described in Mr. G. C. Whipple's "Microscopy of Drinking Water," an invaluable book for waterworks authorities interested in the cause of the odors and tastes which sometimes occur in surface waters. Mr. Whipple classifies these odors and the organisms producing them under three heads as follows:

Group.	Organism.	Natural Odor.
Aromatic..	Diatomaceæ	
	Asterionella.....	Aromatic—geranium—fishy.
	Cyclotella.....	Faintly aromatic.
	Diatoma.....	" "
	Meridion.....	Aromatic.
	Tabellaria.....	"
	Protozoa	
	Cryptomonas.....	Candied violets.
	Mallomonas.....	Aromatic—violets—fishy.
Grassy.....	Cyanophyceæ	
	Anabæna.....	Grassy, mouldy—green corn—nasturtiums.
	Rivularia.....	Grassy, mouldy.
	Clathrocystis.....	Sweet, grassy.
	Cœlosphærium....	" "
	Aphanizomenon...	Grassy.
Fishy.....	Chlorophyceæ	
	Volvox.....	Fishy.
	Eudorina	Faintly fishy.
	Pandorina.....	" "
	Dictyosphærium...	" "
	Protozoa	
	Uroglena.....	Fishy and oily.
	Synura.....	Ripe cucumbers—bitter and spicy taste.
	Dinobryon.....	Fishy, like rockweed.
	Bursaria.....	Irish moss—salt marsh—fishy.
	Peridinium.....	Fishy, like clam shells.
	Glenodinium.....	Fishy.

In this table it will be noticed that after certain organisms there are several odors, separated by dashes. This signifies that as the numbers of the organisms increase they impart different odors to the water. Asterionella, the most pronounced of all, gives the water a slight aromatic odor when a few organisms oc-

cur; as the number increases the odor resembles that of rose geraniums, and if the growth continues, the water has a nauseating fishy odor which is very disagreeable. The trouble with the water supply of Brooklyn late in the summer of 1896 was due to this organism. It did not occur in sufficient numbers to cause any trouble in the wells and ponds from which the supply is drawn, but developed rapidly in some of the shallow basins used as distributing reservoirs. The trouble has been remedied by building a by-pass around the basins through which the water passes without access to the light during the season when the organisms are liable to grow. Some of the organisms give off still different odors when decaying, such as the pig-pen odor of decomposing cyanophyceæ. Enough has been said, however, to show that the numerous cases of trouble from odors can be traced in most cases to a few species of organisms. It will take a biologist but a short time to determine which species is responsible in any given case. Unfortunately, it is not such a simple matter to devise a satisfactory remedy, and in the present state of knowledge on this subject, no general advice of any value can be offered.

CHAPTER VII.—GROUND-WATER SUPPLIES.

The growing importance of ground-water supplies in this country makes it important to go into the classification of such supplies in some detail. In Great Britain and in Europe the utilization of underground water has been studied with much greater care than on this side of the Atlantic, for the reason that unpolluted surface supplies are relatively rarer there than here, and the American engineer will find many instructive examples of springs and wells in use in foreign water-works.

Two broad classifications of ground water may be made at the outset. The first is the water which filters from a river or pond into the soil forming its basin. This water generally retains many of the characteristics of the source from which it is derived, although it is often possible to select a point from which a supply may be drawn of much greater purity than the river or pond water, owing to the fact that the quality of the latter has been improved during its passage through the earth. The second great class of ground waters includes the water which has entered the ground from a variety of sources, but has been checked in its downward percolation by more or less impervious strata. If the water has merely settled down vertically through a single stratum, it is generally possible to reach it with a shallow well. In case the underlying impervious stratum is inclined, the water will flow down along its upper surface until, possibly, the pervious stratum in which the water is confined is overlaid in turn by another impervious stratum, when it will be necessary to sink a deep well to secure a supply. In case the inclined pervious stratum finally opens to the air, it will lose its water by a spring at the outcrop. Sometimes all the strata are inclined for a long distance, so that the water in the porous bed at the bottom of the incline is under considerable pressure. If the distance from the surface of the ground at this place down through the impervious strata to the water-bearing rock or earth is much less than the difference in

elevation between the point where the water entered the earth and the foot of the inclined porous stratum, then a well sunk at the latter point will be what is called an artesian well, and the water will rise in it above the surface of the ground. This term, artesian well, is often used incorrectly to designate a deep well. The different classes of well are shown in Figure 15.

METHODS OF COLLECTING GROUND WATER.

With regard to the methods of collecting ground water, it is evident that no one plan is applicable to all cases, but the follow-

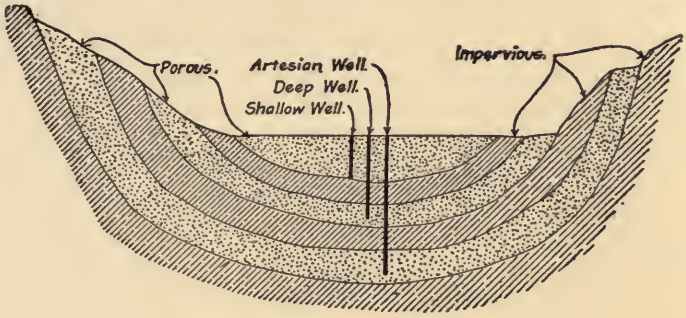


FIGURE 15.—DIAGRAM OF WELLS.

ing general statements, largely based on Mr. Stearns' paper on the "Selection of Sources of Water Supply," are necessary as an introduction to more detailed descriptions. Where the material is coarse and porous within a short distance of the surface and the quantity of water required is not very large, a circular well, covered to exclude the light, is generally the best. In other instances, where the material for a long distance from the surface is impervious, but is underlaid with pervious material, it is impracticable to excavate a large well to the required depth, and it becomes necessary to sink tubular wells to the porous stratum which may sometimes be found overlying the rock.

Tubular wells may be connected by means of a large suction pipe directly with the pump, which is generally the cheaper form of construction, or they may be connected with excavated wells or filter galleries, into which the water from the tubular wells will flow.

Filter galleries are built in some instances where it is desired to intercept the ground water from a greater area than will be in-

fluenced by a single well. They are, in fact, elongated wells, which, however, are not usually sunk to as great a depth. Filter basins perform the same office in collecting water as wells and filter galleries, but being uncovered, the water in them deteriorates owing to the rapid growth of vegetable and animal organisms which flourish in this kind of water when the light is not excluded. This form of construction should therefore be avoided.

There are many instances in which the main supply comes from wells and filter galleries, but is increased by means of driven wells extending from them into porous strata at lower levels.

In developing springs it is generally customary to make a sort of masonry well about them, care being taken that the work interferes as little as possible with the flow. Sometimes tunnels are driven into slopes from which springs issue, in order to secure more water than a natural hillside spring will furnish.

In the case of very deep or artesian wells, it is usually necessary to employ a well-sinker's outfit, or to let the work to contractors accustomed to the operations. The latter plan is generally the cheaper and more satisfactory.

QUANTITY OF GROUND WATER.

The amount of water that may be obtained from deep and shallow wells is so often overestimated that it is necessary to call attention to the fact that the quantity available depends on the same conditions as the amount of surface water, the extent of the catchment area, the rainfall, the proportion of the rainfall entering the ground and the capacity of the basin to hold ground water. There is also in this case an additional condition, which is the porosity of the soil or its capacity for storage. It is manifestly impossible to obtain more water from the ground than enters it, and hence it is frequently of advantage to estimate the ground water in the same manner as a surface supply. Mr. F. P. Stearns, who studied this subject carefully in connection with his examinations of the water-works of Massachusetts, has given the following outline of the method of procedure:

"It is sometimes feasible in the case of a proposed ground-water supply to determine whether the source is worthy of a careful investigation by means of the table (see page 14) for determining the volume of surface water obtainable with different amounts of storage. If, for instance, it is desired to obtain a supply of 300,000 gallons a day, and the watershed draining toward

the proposed well is 1 square mile, we find from the table that in the case of a surface water supply the storage, when there are no water surfaces, must be 29,800,000 gallons. If the supply is to be taken from the ground it seems fair to assume that at least an equal amount of storage will be required; and the question to be considered relates to the probability of obtaining this amount of available storage, which is equivalent to the contents of a pond having an area of 10 acres and a depth of 9 feet. Porous gravel or sand when saturated contains in the neighborhood of 35 per cent. of water, but of this a portion remains after the ground is drained, so that only about 25 per cent. of the whole mass will run out when the water table is lowered. Therefore, in order to obtain 300,000 gallons daily from a square mile during the driest period, it is necessary to have a storage equivalent to that furnished by 40 acres of porous gravel in which the water table can be lowered 9 feet. A superficial examination of the ground may show whether it is probable that this amount of storage can be obtained and in this way indicate whether it is desirable to make further investigations."

The presence of water near the surface may be tested by sinking a tube well and pumping from it by hand, if the quantity wanted is small, or by means of a small portable steam plant if the works are to be of a fair size. The water may be measured with sufficient accuracy by letting it flow over a small weir at the end of a plank trough. Small centrifugal pumps are generally used on these tests. It is desirable to sink several tubes separated by distances depending on the character of the soil, so that the fall of the ground water due to pumping from one well can be measured in the others. In French and German textbooks on water supply, there are often elaborate mathematical investigations of the yield of tube wells in various soils, but while these may be of theoretical interest, they are of very little actual value. It is such an easy and inexpensive matter to make the necessary tests for a small plant, that there is no excuse for failing to do so, especially as the results of such an examination frequently indicate that the best location for the wells is not shown by surface indications.

One great advantage of thorough tests of this nature is that they show the nature of the water-holding basin. This is a very important matter, for it may happen that an apparently abun-

dant supply will suddenly give out, owing to the water being drawn from a large basin with a small tributary catchment area. For a year or so such a basin may furnish an ample supply, but after its water table has been drawn down the catchment area is too small to meet the draft. It is believed that the partial failure of the first well of the Peoria Water Company was due to this cause; at first it furnished an apparently unlimited supply of excellent water, but after a few years it began to dry up. There is no apparent chance for the silting of the well. The late A. F. Noyes stated that he kept a 6-inch centrifugal pump running at its full capacity day and night for several weeks to drain a coarse gravel in which a sewer was being laid. It was about a month before any appreciable drop in the level of the water was obtained, but when the pumps finally made an impression, the basin gave out suddenly and the water did not recover its level for a long time.

Where deep or artesian wells are necessary it is evident that a careful geological examination of the country is necessary. In those States which have supported geological surveys, valuable information can often be obtained from their officers or their published reports, and the United States Geological Survey can sometimes supply desirable data; in fact, the maps of the last body are of great value to the water-works designer. Deep and artesian wells generally draw water from inclined strata passing under many townships, and an investigation in the neighborhood will sometimes enable an engineer to form a pretty close estimate of the distance it is necessary to sink a well in order to reach a supply of water. There is always a chance of faults and other geological accidents in the neighborhood, however, and hence everyone looking for an underground supply must bear in mind that the search is not certain to be successful.

CHAPTER VIII.—THE UTILIZATION OF SPRINGS.

One of the characteristic general features of French and German water-works, as compared with American practice, is the development and utilization of springs as sources of supply for small communities. In those countries, as previously remarked, the large population and the proprietary rights in streams and ponds tend to compel engineers to study ground water with much care, and it is worth the attention of American engineers to examine the foreign practice in this respect. This chapter is intended to present a very brief summary of the methods in vogue abroad to utilize springs, and is largely based on Herr Lueger's

FIG. 16

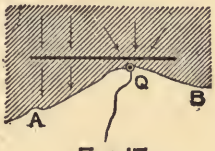


FIG. 17

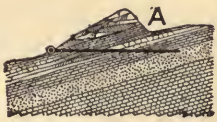


FIG. 18

FIG. 19

voluminous work on the water supply of cities, to which reference has previously been made.

In case an underground stream feeds several springs, which is not rarely the case, the development of one spring and the interception of the water in the others by means of a tunnel or trench will often increase the yield of the first spring at the expense of the others. This plan is frequently adopted.

If it has been determined that a spring Q, Figure 16, starts from a plane of stratification sloping in the direction of the ar-

rows in the cut, it is evident that a large part of the water flows past the spring. The latter, indeed, owes its existence to the recess of the hillside in the neighborhood of Q. In case a tunnel is driven into the hillside over the impermeable stratum, as indicated by the heavy line, the yield of water will be greatly increased, the amount of the increase depending on the area of the surface supplying water to the portion of the porous stratum cut by the tunnel.

Much the same conditions exist in the case shown in Figure 17, which is generally somewhat common. In this cut Q is the spring, A Q B the outcrop of a stratum, of which the arrows show the slope. As a rule no water is visible along the lines A Q, B Q, which are usually hidden by a mass of loose rock or by a meadow which shows the presence of water by its dampness. The threads of water can be intercepted by a ditch or tunnel located as shown, which will make the spring of useful size.

An increase in the delivery of a spring is, moreover, always possible in cases where the pervious and impervious strata follow in regular succession, as indicated in Figure 18. The hillside tunnels of the Oakland, Cal., works are excellent examples; see "The Engineering Record" of June 15, 1895. In this case a tunnel is driven so as to pierce the alternating water-bearing strata, and the result is especially satisfactory if at A there is a fairly level valley with considerable soil from which water can be drawn. It is self-evident that in case the single tunnel does not furnish the full supply desired the yield can be increased by driving branch tunnels on either side in the porous strata, thus intercepting a greater portion of their delivery in the manner illustrated in Figure 17.

In alluvial, glacial and similar beds, it often happens that thin sheets or tongues of barely permeable clay and marl are encountered. In case a very small spring is found at the outcrop of such a sheet, while the area of the catchment basin would indicate the probability of more water being available, it is advisable to search for artesian or "living" water. Figure 19 shows such a clay tongue in a gravel bed. It is apparent that by boring at the site of the spring the water in the reservoir below the clay will be tapped, and it is possible that it will rise with considerable force so as to form an artesian well on a small scale. It is in such formations that "living" or "bubbling" springs often occur; these

are the overflows of a large ground-water system, and supplies of considerable volume can generally be obtained by proper development.

In the development of every spring care must be taken that the water is clear before it enters the outlet pipe, and it is sometimes necessary to provide sand-pits and clear-water chambers in order to accomplish this. For the same reason, the mouth of the outlet pipe should be from 1 to 3 feet above the bottom of the chamber at which it starts and be protected by a screen. The spring ought to be covered by some sort of housing which will keep out leaves, frogs and similar objects while allowing free access for inspection and cleaning. It has been noticed frequently that a spring is affected injuriously if its surface is raised above a certain level, and on that account an overflow of some form is often constructed to prevent automatically such an elevation.

It is well to notice that there are two classes of springs before passing to a brief description of the works developing them, as the latter vary considerably according to the class which they improve. The first are the springs in level tracts where the water rises from a lower reservoir or stream, and the second are the springs on hillsides.

The first has usually a small discharge which can easily be collected in a single basin, and so long as the water level is not essentially changed this method of development has little or no influence on the quantity or quality of the water. If the water level is much lowered, the discharge is frequently increased by a considerable amount, although this is usually done at the expense of neighboring springs at higher elevations.

The case is different, however, in the development of springs which originate at the outcrop of pervious strata on a hillside. With these it is necessary to work into the hillside in order to collect the water, and the development frequently results in a diminution of the original volume of the spring.

SPRINGS IN PLAINS.

If a spring occurs in an extensive level plain, it is generally developed by constructing a basin about it. If the basin is left open, it usually becomes filled with weeds, frequented by frogs, and polluted with leaves and surface water, rendering it useless as a source from which to draw drinking water. Some large open basins, such as the famous Danube Spring at Donaueschingen,

cover such a large area and are so protected by masonry walls and parapets that their water is fairly good. As a rule, however, such springs must usually be provided with a well of masonry and a roof if they are to be used as a source for drinking water, and care must be taken that there is no possibility of polluted surface water fouling them.

The bubbling springs in tracts where no pollution is possible have been utilized in some districts, particularly in the Black Forest. Herr Lubberger has given a description of several such springs, from which the following extract is made:

"In several places, particularly near Loeffingen, I laid bare the surface of the mouth of the spring, and then collected the springs

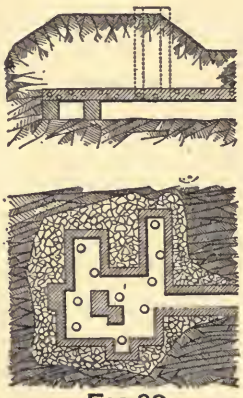


FIG. 20

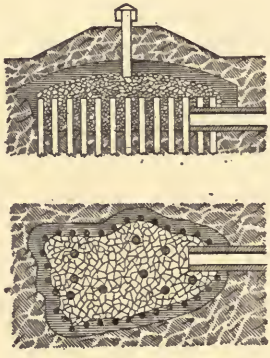


FIG. 21

breaking out at various points by means of a network of tile drains. This network was covered with a concrete slab protecting it from the earth above, which was given a roof shape to shed the rain water; see Figure 20. A ventilating shaft is necessary, of course. In another case, in which I could not lead away the water, I surrounded the excavation with sheet-piling, filled the entire area in which the springs appeared with large stones, and covered this over watertight."

The latter design is recommended highly by Herr Lueger on account of its simplicity and low cost, and he suggests the use of clay as a covering. Figure 21 is a sketch of such a design, which has the advantage of being readily and cheaply opened for inspection and cleaning at any time. Some method of ventilation is

necessary, and he suggests a vertical pipe with a cowl, shown in the sketch, as an appropriate plan.

Elaborate masonry work has been employed at some of the springs utilized by the Paris water-works. Figure 22 shows the chamber of the spring of Armentieres, which is circular in plan and about 32.8 feet in diameter. The very flat dome roof covered with earth exerts a considerable pressure on the wall, which is reinforced by buttresses on this account. The chamber is reached through a doorway, and has a gallery around the interior. There is no overflow, but a drain has been provided for drawing off the water when the chamber is to be cleaned; the doorway affords

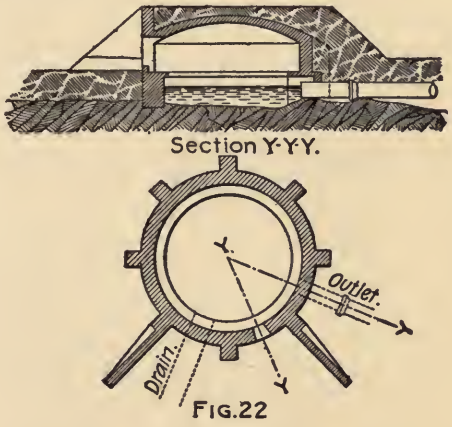


FIG. 22

ventilation. This design has given good service. In some cities the works erected about the springs are of a monumental character, with elaborate entrance towers, as at Lille, for example. Such elaboration, however, does not find favor with many of the continental engineers.

HILLSIDE SPRINGS.

The springs issuing from the side of a hill or mountain have to be treated in an entirely different manner from that followed when the water rises vertically in a plain. In the former case the character of the springs must be studied first. The separate threads of water from which the spring is formed must be sought out, the impermeable stratum must be found, or in case this is far distant, as when the water emerges from a fissure, the immediate cause of the spring must be ascertained.

A spring located at the foot of steep slopes or cliffs is manifestly little exposed to pollution by surface water, and the work of development is little more than the construction of a suitable chamber about and over it. The case is different, however, when the spring rises on a gently sloping hillside and pervious soil lies over the course of the ground water. Such a spring is polluted by the surface water after a protracted rainfall, and it is necessary to make a large area of the overlying surface impervious or to trace the spring back into the hillside until it is some 8 to 10 feet below the surface. When this is done, the various threads of water must generally be intercepted by a tunnel or other work, and the result is often a diminution in the volume of water discharged.

When a spring is followed back into a hillisde in this way, care must be taken that there is no chance of surface water flowing along the tunnel toward the spring, and the entrance must be made large enough for the passage of any materials likely to be needed in repairs or cleaning. Any branch collecting galleries leading into the porous strata should, if lined, have a clear cross-section not less than about $5\frac{1}{4}$ by $2\frac{1}{4}$ feet. This section allows a light wooden working platform to be laid on permanent iron supports when it is desired to clean the channels. In the galleries built at the Vannes springs of the Paris water-works, the platform is a permanent masonry structure which interferes with the cleaning of the channel below. It is also necessary to make some provision for a circulation of air in the spring chamber in case this cannot be secured through the entrance; a ventilating pipe with a hood or cowl running up from the highest part of the roof is the simplest method in most cases.

Every outlet pipe from such a spring should be provided with a strainer at its mouth, and be some distance above the bottom of the chamber, so that the water entering it will be free from solid matter. In case the strainer becomes clogged, or the gate is closed in the outlet pipe, the water will back up in the chamber, and some form of overflow, such as a weir or pipe, should be provided. It is, of course, desirable to have some means of determining the flow of water from the spring in case it is of considerable size, and for this purpose the chamber or the galleries leading to it may be arranged so as to permit the construction of temporary or permanent weirs. Another plan is to determine the capacity

of the basin of the chamber between two different elevations, and then find how long it will take the spring to raise the water surface from the lower to the upper level; from this information it is a simple matter to calculate the discharge. In case the water contains much sand, as springs flowing from sandstone formations are liable to do, it may be necessary to construct a sand-pit like those used on the headworks of plants drawing supplies from mountain streams. The sand-pit is simply a long, comparatively deep and wide basin, in which the velocity of the stream is checked sufficiently to cause all suspended matter to settle to the bottom.

The application of these principles is best shown by studying a design embodying them. Figure 23 represents a small spring chamber designed by Herr Lueger. It is walled, arched and cov-

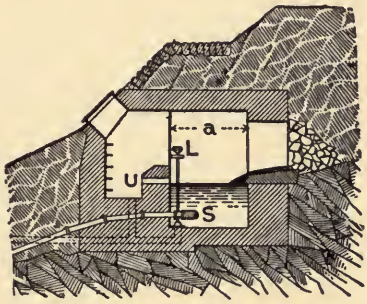


FIG. 23

ered with earth. It is accessible through a sharply inclined entrance furnished with two folding doors or a removable cover. Within the entrance is a shaft for descending to the basin and thus reaching the spring. This shaft is drained by a pipe indicated by the dotted lines, which leads away all surface water that may enter through the opening above; this pipe also serves as an outlet for the overflow U of the basin. A drain pipe for drawing off the water in the basin joins the same pipe, and can be opened and closed by a plug fitted with a handle. The main outlet pipe has a cylindrical screen S over its mouth. The opening between the shaft and the main chamber can be fitted with doors in case it appears necessary to guard against frost or heat. The ventilation is amply provided for by the entrance shaft. The water from the spring passes through an opening in the rear wall, which

is backed by large stone and gravel to keep back the sand. In case the latter enters the chamber in an appreciable amount the dimension *a* of the chamber should be increased so as to give a basin of sufficient size to form a sand pit. It is assumed that the spring chamber can be somewhat sunk into the impermeable stratum. If this condition cannot be filled the spring must be collected in a pipe or other channel, and the chamber must be covered somewhat deeper or sufficient protection against infiltration provided otherwise.

The criticism may be made against this chapter that it advocates antiquated and costly principles and methods, but the writer has already stated as clearly as he can that these underground sources of supply are at best something of a gamble, and are not to be employed when good surface supplies are as readily secured. On the other hand, American engineers have not practised the development of ground water sources to the same extent as their colleagues across the Atlantic, and in view of the successful new works, especially small works, which are being constructed abroad for the utilization of such sources by the means outlined in this chapter, or by others, to be described later in connection with wells, it is believed that an important feature of water-works design would remain unmentioned without these notes.

CHAPTER IX.—OPEN WELLS.

The two classes of open wells which have been used for water-works purposes are those of large diameter and comparatively shallow depth, which include most American wells, and those of small diameter but considerable depth usually extended downward by a bore hole or driven well. This latter class has been developed mainly to obtain water from the chalk formations of England, although it is coming into favor in Germany.

Large shallow wells sunk near rivers or lakes are liable to furnish a supply of genuine groundwater more or less mixed with less desirable water from such sources. When such a well is in regular service, the profile of the ground-water surface, as shown on a vertical cross-section through the well, approaches that of a parabola. The curve is called in theoretical investigations a depression curve, and its form varies, of course, with the draft from the well, the nature of the walls of the well and surrounding earth, the available quantity of ground-water, and the slope of the water table, or top of the ground-water, before the construction of the well. While the theoretical investigations of these curves and surfaces have no immediate practical application, it is of value to determine their form by a combination of theoretical and experimental methods, if it can be done without too great an expenditure of time and money, as the knowledge thus gained is useful in estimating the effect on the well of neighboring bodies of water. It has been determined experimentally that the depression-curve of even small wells may begin anywhere from 200 to 400 feet from the well. Hence the natural fall of the groundwater of a large area will be changed by such a well, and there will be a tendency to draw water toward it from any neighboring stream or pond within this area, although under normal conditions there would be no such tendency.

Wells of this sort may be divided into two classes, according as the walls are tight or pervious, the latter being comparatively rare

in this country. The wells with tight walls and no bottom are used where the water is obtained from a stratum through which the water moves freely. The proper diameter to give such a well to enable it to yield a maximum supply depends upon a number of factors. Where the water-bearing stratum is fine sand, a marked increase in the yield can usually be secured by enlarging the diameter of the well, but this is not the case with coarse gravel and the like. The main advantage of a large well, however, is that it acts as a reservoir, preventing great fluctuations in the water level and consequently in the depression curve. Regularity and uniformity in the draft from such wells is very desirable, and the reserve supply of water in a large well is useful for attaining this condition. Large wells possess the further advantage that the rate of the flow of water into them is less than with small wells, and there is therefore less liability of particles of sand and earth being displaced by the passage of the water. This consideration is of less weight where gravel furnishes the water than where sand occurs; it is possible that the sanding of many wells might have been prevented had attention been paid to this feature. As a matter of record, the following figures are quoted here from Herr Thiem's description of the water-works of Nuremburg, as to the vertical velocity of water in inches per second which will keep floating particles of sand of the sizes (in fractions of an inch) mentioned:

Size	0.010	0.010-0.020	0.020-0.039	0.039-0 079	0.079-0.118
Velocity	1.14	1.38 -2.72	2.95 -3.78	4.33 -6.69	7.08 -32.28

The slight velocity necessary to take the smaller sizes of sand into suspension indicates that where a well must draw its supply from a stratum of such sand, the bottom should be covered with coarse sand and gravel in graded sizes, which will act as a sort of filter to hold back the fine particles when the velocity of the ground-water stream is great enough to displace them. This principle was used many years ago by the late William Gill in constructing wells with pervious walls, and various modifications of his plans have since been adopted. In the more recent wells of this class, the walls, which have numerous openings in them, are surrounded by layers of sand and gravel of graded sizes, the coarsest being next the walls and on the bottom of the well.

The construction of small wells is comparatively a simple matter, especially when they are to be lined with brickwork. Form-

erly it was customary to sink them like open bridge caissons, using a timber curb on which the brickwork was built up as the well was excavated. The friction of the earth against the masonry frequently prevented the latter sinking as intended, and another well would then be started inside the first and carried down in a similar manner until the weight of its masonry likewise proved insufficient. This method of sinking may still be used to some extent, but it has been abandoned generally in favor of the more rapid method of underpinning. This consists of sinking a shaft as deep as seems safe, and then lining it with masonry started on a wood or iron curb. The shaft is then sunk a few feet farther, and the masonry built up from the bottom to join that already laid. Various plans are adopted to prevent the masonry from falling when the earth below it is removed, which is not liable to occur in most places if the upper courses are backed with well-rammed earth, on account of the great frictional resistence of such a shaft. Sometimes three courses of brickwork are laid in cement every 5 feet or so to form stiff rings, and sometimes iron curbs are laid in the lining and hung by ties to beams higher up in the well. Another plan is to use raking props to support the existing lining. In this last case, after one section of the lining has been completed, a small shaft is sunk to a depth of a few feet, and a footing block bedded firmly on the bottom. Then slanting trenches are cut in the earth so that struts can be placed between the curb and the block. As soon as these timbers are wedged into place the masonry is generally held well enough to enable the earth to be excavated for the next course, which is laid on a curb like the first, this method of working being continued until the desired depth is attained. It is customary to lay the upper part of the masonry in good cement mortar so as to prevent surface water from filtering into the well, but otherwise only occasional courses are laid in cement, for the sake of stiffness, as before mentioned. The methods employed in sinking large wells are many, as the following paragraphs will show:

The brick well of the Rhinelander, Wis., water-works is 30 feet in diameter, and was sunk through loam, clay and gravel. The curb was formed of eight thicknesses of oak plank. The bottom two courses were about 6 inches wide, the next two 8 inches, the next two 12 inches and the top two were 16 inches. The outside of the curb was formed by a ring of vertical oak plank measuring

2 x 8 x 63 inches. They were spiked to the horizontal courses so as to form a sort of cutting edge about 8 inches below the latter and also a backing to the lower courses of the brickwork. This was started on top of the curb as a 16-inch wall 3 feet high, on top of which a sill of two courses of timber was placed. This sill was connected to the lower one by anchor bolts running through the masonry, and the outside ring of vertical plank was spiked to it as well as to the curb. The main wall of the well was built on the sill, and its weight served to sink the whole mass gradually as the earth below the cutting edge was removed by means of pick and shovel.

The well at Webster, Mass., has a clear diameter of 25 feet at the bottom and 26 feet at the top and is 30 feet deep. It was sunk in sand and gravel previously tested by means of ten 2½-inch tube wells, and is located about 300 feet from a large lake. Mr. Frank L. Fuller, M. Am. Soc. C. E., engineer of the works, reports that the excavation was made by driving 2-inch plank sheeting outside of ribs made of three or four thicknesses of 3-inch spruce plank sawed to the proper radius and spiked together. The sheeting was in two courses, as the well was too deep for one. The water encountered during the excavation was removed by two 6-inch centrifugal pumps, the yield being about 1,000,000 gallons in 24 hours at a depth of 30 feet. After the excavation was completed, a 5-foot dry rubble wall was started, the thickness being reduced at a height of 3 feet to about 3 feet 8 inches by an inside offset, which was used as a support for a 12-inch brick lining laid in cement. The rubble backing was gradually reduced in thickness to about 2 feet, the last 4 feet being laid in cement, and protected by heavy coping stones which bind the stone and brickwork together and make a foundation for the wooden roof of the well. The cost of this well was $13,190.

In sinking wells in sand containing a large amount of water, it may prove advisable to make no attempt to keep the water out of the well. If strong pumping is necessary to keep the bottom free enough for the men to work, the material in the neighborhood of the well may be affected injuriously. If it is believed this will be the case, the core of the well must be removed by some method of dredging. One of the simplest of these methods is the use of the sand bucket, which was first employed in sinking a large well in Brooklyn many years ago and has since been tried successfully

on a number of undertakings. The sand bucket is believed to by the invention of Messrs. John Bogart and C. C. Martin.

It is a sheet steel cylinder about 3 feet long and 18 inches in diameter, open at the bottom and closed at the top, to which a long handle of 3-inch pipe is attached. The top also has an air valve which can be closed by means of a line. The bucket is forced into the sand until it is completely filled and all air driven out through the valve. The latter is then closed by means of the line and the bucket raised to the surface, its contents being held in place by the pressure of the atmosphere. The apparatus works well in some soils, such as sand, but is not satisfactory in others.

The so-called Indian shovel is another apparatus for removing material by hand from below water. It consists of a long handle, to the lower end of which is hinged a spoon-shaped blade. A chain is attached to a link running out from the sides of the blade and a second chain is attached to a lug riveted to the bottom of the blade at its back. This second chain is kept taut while the blade is forced into the sand or gravel by the handle. When this part of the operation is finished, the chain is loosened and the front one tightened, pulling the blade into a horizontal position so that it forms a small skip or bucket, which is lifted to the surface and emptied.

The sand-jet used in sinking bridge piers might also be employed under certain conditions, although it would generally be an expensive apparatus. Other apparatus will be found described in the catalogues of dealers in well fittings. Occasionally large wells have been sunk by the pneumatic process, in the same manner as caissons for bridges or high buildings. The well of the Basel, Switzerland, water-works was constructed in this manner. The Poetsch freezing process was used at a Michigan plant. Any description of these various methods would be out of place here, however, and the reader is referred to the volumes of "The Engineering Record" for further information.

The construction of deep, open wells, consisting of a long masonry shaft usually terminating in a bore-hole, is a practice that has found little favor in this country, where it is considered cheaper to construct the bore-hole from the surface and dispense with the enlarged upper shaft. The latter has the advantage of acting to a certain extent as a reservoir and in some cases enables the pumps to be arranged more advantageously than in a plain

bored well. The general foreign practice in regard to these wells is given in the following paragraphs. Further information on the subject will be found in the "Proceedings" of the Institution of Civil Engineers, Volume XC., from which Figures 24 and 25 have been taken.

The well at Workingham is 6 feet in diameter for a depth of 200 feet, and is lined with 9 inches of brickwork in cement mortar. For the next 60 feet, the shaft is reduced to 4 feet in diameter and is lined with 4½-inch brickwork in neat cement. For the next 95 feet an 18-inch boring was made and lined with cast-iron pipes 16 inches in diameter. The last 53 feet of the well was an unlined 16-inch boring. The 4-foot shaft was carried 5 feet up the 6-foot shaft and a cement joint made between the two to prevent the entrance of water, and a similar joint was made between the cast-iron lining and the 4-foot shaft. The strata penetrated were quite varied, including 259 feet of London clay, 84 feet of sand and clay of different characters, 2 feet of sandstone and 63 feet of chalk at the bottom. Twelve months were required to sink this well.

Mr. William Matthews, M. Inst. C. E., sank a pair of wells for the Southampton Water-Works in a manner that may prove useful in working hardpan and material of similar nature under water. Owing to the ease with which borings are made in chalk, he decided to sink, instead of one large well, two 6 feet in diameter and 11½ feet apart to a depth of 100 feet from the surface. The upper part of each was enlarged into a pump chamber, resting on a heavy concrete floor. These floors were pierced by two heavy cast-iron cylinders, 6 feet in diameter and 6 feet long, which formed the entrances to the wells and acted as guides to the boring tools.

The boring was done by three chisels of wrought iron with steel shoes, shown in Figure 24. The center chisel, with a plain blade, was longer than the outer ones, each of which had a tongue forged into its outer edge to act as a guide while the apparatus was being raised and lowered, thus preventing the tool from cutting into the sides of the bore hole. The boring rods were 3 inches square and were furnished in 10-foot lengths, which were coupled together by screw joints. The rods and tool were dropped on the chalk and then raised a few feet by means of a steam winch and cable, this process being continued until enough chalk had

been broken up to require removal. The tool was slowly rotated by hand during the work so as to distribute the blows over the surface.

The broken material was removed by a miser or large auger shown in Figure 25. It was constructed of ⅜-inch boiler plate strengthened with substantial angle irons and plates. "Each half of the lower portion was made tapering with a helical twist, and where the upper edge of one plate was brought nearly over the lower edge of the other plate, on either side, a hinged flap was inserted to retain the *débris* entering the tool." The edge of each plate under the flaps was furnished with steel-toothed cutters. "Internally the miser was divided into two compartments, so that in the event of one flap-valve failing to act, the whole efficiency

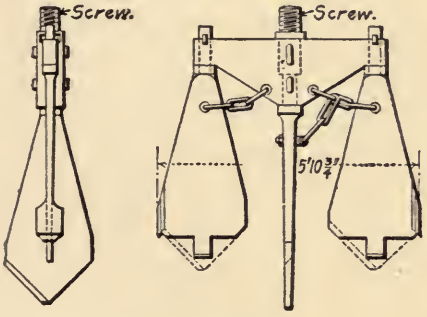


FIGURE 24.—CHISEL.

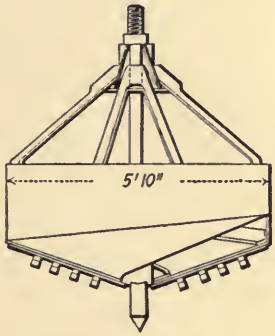


FIGURE 25.—MISER.

of the tool was not destroyed; moreover, the motion of the *débris* entering by one passage did not tend to choke the action of the other." A wrought-iron spindle projected from the bottom of the tool to guide it, and a strongly braced rod at the top enabled other rods to be screwed to the apparatus to form a shaft for turning it around, which was done by men with levers. The miser generally came up two-thirds full. When all the apparatus was in good order the average depth sunk per day was about 5 feet.

The well at Petersfield, England, was started as a shaft 8 feet 4 inches square. Down to 50 feet from the surface, according to Mr. Robinson, M. Inst. C. E., the engineer of the undertaking, the strata were quite dry; at 51 feet small quantities of water ap-

peared, and at 56 and 66 feet respectively two fissures were met which brought in considerable quantities of water, but not enough to supply the town. An adit or tunnel was driven on the course of the upper fissure for a distance of 53 feet, but this did not increase the yield as much as was anticipated. The lower part of the shaft, which was in sand, was enlarged to form a storage chamber 16 feet in internal diameter and lined with brickwork, the floor being 67 feet below the surface. A bore hole was then put down to a depth of 96 feet from the surface, and at this depth a large volume of water was obtained from a sand stratum. The water rose in the chamber 4 feet higher than before, and the total supply was increased to about 7,200 gallons an hour.

The construction of adits running out from wells, as mentioned in the last paragraph, is a method of increasing the yield of large wells which deserves attention. It is, of course, necessary to drive them at right angles to the direction in which the ground water is flowing. Although the method has never been tried, so far as the author is aware, he is of the opinion that horizontal tubular wells with perforated sides might be driven advantageously by water jetting or some similar process from the sides of shallow wells of large diameter where the increased draft threatened to silt up the bottom.

CHAPTER X.—DRIVEN WELLS.

A driven well is "one formed by driving or forcing a wrought-iron or galvanized-iron tube down into the stratum from which the water is to be taken. These pipes are generally from 1½ to 8 inches in diameter, and may or may not be furnished at the lower end with a wrought-iron or steel point. Above this point the pipes are perforated for some distances with holes to admit the water."

Wells of this type from 60 to 100 feet deep were driven in 1849 and 1850 by E. W. Purdy, a Milwaukee well-maker, and in 1859 Calvin Horton drove wells of the same sort in East Somerville, Mass., and neighboring towns. In 1867 or 1868 he began to connect a system of wells to a single suction main leading to a steam pump. When the British-Abyssinian campaign was planned in 1867, the question of water for the troops was a difficult one to solve. Finally Mr. J. L. Norton modified the American driven-well practice for military purposes, and his apparatus was used so successfully during that campaign that driven wells using small tubes are frequently called Abyssinian wells in Great Britain and on the Continent.

No better general description of driven-well work has come to the author's notice than one prepared by the late Albert F. Noyes, M. Am. Soc. C. E., who had much experience with these wells. The best results "are usually obtained by driving an open-end pipe having attached to its lower end a steel shoe similar to a common pipe coupling. This is first forced into the ground and a small hollow rod or pipe, to which a steel point is welded, has small holes drilled into it so that a jet of water forced into the rod will discharge into the holes and play on the point of the drill. This loosens the material about the point of the tube, and a large part is forced, by overflow, out from the pipe. The wash pipe is then removed, the tube driven until checked by the resistance of the earth, when the process of cleaning out is renewed until the level from which it is desired to take the water is reached.

"When a number of water-bearing strata of coarse material are encountered, additional pieces of perforated pipe may be advantageously inserted, thus materially increasing the yield and diminishing the velocity of the water flowing into the pipe at any one point, thus preventing a tendency toward an inflowing of the finer material.

"When it is desired to determine the character of the strata through which the pipe is being driven, a sand-pump may be used with better results than a driving-rod. This is usually a metal cylinder of a little less diameter than the inside of the well tube, having a clap valve at one end and a handle at the other which acts as a guide to a rod to which is attached a washer closely fitting the inside of the cylinder. The rod is worked up and down, thus filling the cylinder or pump with the material which is withdrawn. The process is continued until the well is cleaned out; after this the driving continues.

"The driving of small tubes is usually effected by sledges striking on an iron-bound wooden block, or by a small, portable pile-driver, or by a weight running over a portion of the tube and striking against an iron clamp. This weight may be raised by a rope passing over a sheave or sheaves attached to a tripod, or clamped to the upper portion of the tube. The larger tubes are driven by pile drivers with large, heavy wooden hammers attached to a rope which passes over a sheave at the top of a high derrick, and is operated with steam power. In driving, great care must be taken to keep the joints of the pipe well screwed up, for the jar from driving has a constant tendency to unscrew the sections of the pipe."

The principle on which driven wells operate has been explained very clearly by Mr. Wynkoop Kiersted as follows: "It is a well-known fact that whenever water is continuously drawn from a well or system of wells, a hydraulic slope of the ground-water is established, falling toward the well or center of draught, the profile of which in any vertical section is an irregularly curved surface. The head which induces and maintains the flow into the well, regardless of that causing the natural flow through the ground, is the difference of level between the natural surface of the ground-water and that in the well. This difference of level in a system of driven or bored wells, whether of the single or double tube kind, depends upon the vacuum produced by the pump in the

suction pipes, and it naturally follows that the nearer the pumps and suction pipes are placed to the natural level of the ground-water, the less the frictional resistance in the suction pipes, the greater, for any given vacuum, will be the static head causing the flow of water into the wells. Now, the resistance of flow through the voids of the water-bearing sands is too great to allow any of the available head to be consumed in unnecessary resistances in the suction pipes and in the pump lift above the natural surface of the ground-water; therefore, in my opinion, it becomes essential to locate the pumps and suction pipes very near or even below the natural level of the ground-water at the time of construction, and to proportion the size or sizes of the suction pipes so that there will result but little friction loss."

Theoretically the diameter of a driven well with perforated sides does not have an appreciable influence on the yield, but for practical reasons it is necessary to have at least a certain extent of perforated surface. If this amount of surface is not provided, the velocity of the water entering the well will exceed the limits mentioned in connection with open wells as sufficient to take sand of various sizes into suspension. In case the thickness of the water-bearing stratum is known, this principle enables an estimate to be made of the proper diameter of a well to yield a definite amount of water. Another factor which some foreign engineers employ in estimating the maximum amount a given well should yield is the greatest permissible depression of the surface of the water, which they fix at 8 to 9 feet. Another very weighty influence in selecting the diameter of the pipe is the greater or less difficulty of sinking the various sizes with the appliances at hand. It is not advisable to choose small tubes for wells in gravel and coarse sand, but in case fine sand is penetrated a large number of small tubes will give the best results for a given outlay.

SINKING WELLS.

In sinking wells in fine sand, it is often the custom to cover the screen with finely woven wire netting as an additional precaution against the entrance of grit. Sometimes netting is advantageous, and sometimes it clogs up so quickly as to require cleaning in a short time. This work can be done in several ways, by blowing high-pressure steam down the tube, by forcing water down it by means of a pump, and by pulling up the tube and cleaning the network by means of brushes, jets of water or other appliances.

In driving wells in very fine sand, it is sometimes advisable to take special precautions against the entrance of the sand into the pipes. One ingenious plan, capable of various modifications, was described in a paper read before the Society of Engineers by Mr. Robert Sutcliff on the use of tube wells for large water supplies. A tube well was driven at Chislehurst into an extremely fine sand, and it was found impossible with the finest strainer to obtain any supply of clear water. The tubes were withdrawn, the point screwed off, and the open pipe driven into the same hole. The pump was then attached again, and four or five barrow loads of sand pumped up. Before doing this, however, six barrow loads of good, clean, sharp gravel were brought to the spot. The pump was removed, and down the tube, which was 1¼ inches inside diameter, as much gravel was rammed as was needed. The open tube was then withdrawn, and a pointed and perforated tube driven into the gravel bed thus formed. A sand tube was then dropped into the well to keep back the grit, and upon again attaching the pump the water came freely and cleared rapidly.

The so-called sand tube of English engineers is essentially two concentric perforated tubes. About the perforated portion of the inner tube enough horsehair cloth has been wrapped to fill completely the space between it and the outer tube.

At Orpington what is known as a blowing sand was dealt with somewhat similarly. Owing to the nature of the sand a cavity could not be made as in the previous case. A hole 6 or 7 inches in diameter was therefore bored and piped down with large tubes until several feet of the quicksand had been passed. This quicksand was removed from the pipes with an ordinary boring shell and gravel was rammed down, the large tubes being gradually withdrawn as the work progressed. A small 1¼-inch tube was then driven into this vertical gravel bed, and a well yielding 240 gallons an hour thus obtained.

The natural discharge of small driven wells without pumping may be measured while they are in use by means of a modification of Pitot's tube devised by Mr. A. O. Doane, of Newton, Mass. "This consists of two lengths of ¼-inch brass tubing about 12 feet long connected at the top by means of a inverted U-shaped glass tube, with an air cock at the top, and the whole fastened to a board for convenient handling. (Figure 26). The board is cut off just above the ends of the brass tubes. One tube is open at

the lower end, and the other is closed and has a fine opening on the side. To use these tubes they are inserted in the well so as to extend down below the T branch connecting into the 24-inch main; an air pump is connected with the air cock and the water raised until it appears in both branches of the U. On closing the cock, the water drops slightly in one of the branches and the difference in level in the two branches is the head under which the water is flowing, or the value of h in the formula $v = \sqrt{2gh}$. These tubes were tested with a known flow of water, and found to give results within 5 per cent.; all joints must be perfectly air tight." If the velocity of the upward flow of water is found in this manner, the discharge is readily obtained by multiplying the area of the pipe by this velocity, care being taken that all the measurements are expressed in the same unit, preferably feet.

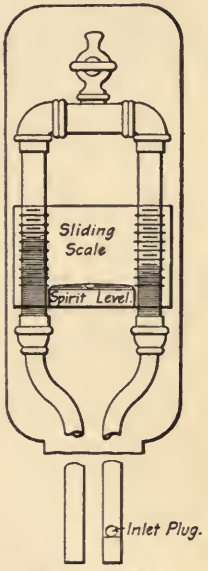


FIGURE 26.

The tube wells at Plainfield, N. J., are 6 inches in diameter and driven from 35 to 50 feet into a bed of water-bearing gravel. The wells were not driven at right angles to the direction of the underground stream, and on this account but partially intercepted the flow. Inside each 6-inch tube, which is perforated at the bottom in the usual manner, there is a 4½-inch suction pipe 25 feet long. This inner tube is attached at its upper end to a casting fitting on top of the outer tube, a rubber gasket being used to make the connection tight. The casting has a vent on the side through which air is admitted to the space between the two tubes, thus converting the well into an open well so far as the work of pumping from it is concerned. The top of the casting is also provided with a tap by which a vacuum gauge may be applied. The wells were connected with a wrought-iron suction main from 8 to 12 inches in diameter. The most distant well is 500 feet from the pumps, and "shows in an interesting manner by the vacuum at the well head and increased vacuum at the pump, the effect of long suction and friction in the main." This is indicated more clearly by some figures in a description of these works by Mr. L. L. Tri-

bus, M. Am. Soc. C. E., in Volume xxxi. of the "Transactions" of the American Society of Civil Engineers. During a 24-hour test the average suction lift in feet, referred to the level of the upper pump valves, was 21.78 at the farthest well and 22.83 at the nearest, the suction in the air chamber being 26.16.

This plant, like many others, was constructed with the suction main and pumps too high. During a season of long-continued dry weather the water level became so low that difficulty arose with the extreme suction lift obtained, from 20 to 28 feet, according to the rate of pumping, a fall of some 6 or 7 feet since the earlier observations, so it was deemed best to lower the pumps 8 feet 1 inch below their first positions.

This trouble occurs so often that special attention should be paid to avoiding it in making plans for the work. The usual objection to setting pumps and mains low is that the ground-water makes the work of excavating difficult, but this can be overcome by strong pumping from the permanent wells, and, if necessary, temporary ones driven in such a manner as to drain the site of excavations.

The methods of sinking driven wells are numerous, as is evident from what has been said, and contractors vary in their preference of plans for work in the same locality. Perhaps the most striking instance of this is furnished by the history of the Lowell, Mass., wells, which is here condensed from the annual reports of Mr. George Bowers, City Engineer:

The first work of this nature was the sinking of seven 3-inch wells. At the upper end of a wrought-iron pipe, 19 feet long, a hose was attached, which was connected with a pump. A strong stream of water was forced down the pipe by this means, cutting away the earth so as to allow the pipe to drop gradually. The water and excavated material came to the surface outside the pipe. When the first length had been driven in this manner, a second was coupled to it, and the process continued until rock was reached. The pipe was then pulled up to allow water to enter its lower end, and it was finally connected with a pump, which was run several hours, or until the water was clear, when the well was ready for testing. Samples of the earth washed up were preserved, together with notes showing the depths from which they came.

The second set of wells, which were 3 inches in diameter, had

perforated lengths covered with brass gauze, and were provided with a steel shoe at the lower end of each screen. Such a combination was driven a short distance by means of a 150-pound drop-hammer, and the earth inside washed out by means of a pipe connected with a force pump. This operation was repeated every 5 feet until the well was driven. A dry-wood plug was then swedged into the steel shoe.

The third plant consisted of 6-inch wells, which were sunk by means of a sand-bucket or sand-pump nearly as large as the inside of the pipe and about 6 feet long. Such a bucket is merely a cylinder with a valve opening upward at the bottom. It is churned up and down by wooden rods connected with screw couplings until it is filled, when it is lifted up to the surface and emptied. A large pyramidal frame like those used in oil well sinking was employed at Lowell. The bucket removed stones 5½ inches in diameter, when wells were driven through gravel. While the bucket was in use, the pipe was gently turned or rocked, and sunk by its own weight in sand; in very hard gravel it was sometimes driven a little with a heavy wooden hammer. Where stones too large to come up through the pipe were encountered, they were broken with a large drill, or, if this was impossible, the pipe was pulled up and the location of the well abandoned.

Two push wells, lying almost horizontal under the surface, were also sunk by this contractor. They were driven by means of a strong dam or bulkhead to press against, a strong clamp on the end of the pipe, and four jackscrews between the bulkhead and clamp. This proved a slow and expensive plan, but enabled a very long screen to be placed in a thin water-bearing stratum. Although it has worked satisfactorily in other places, the additional yield of water did not pay for the difference in cost at Lowell.

After sinking the pipe the desired depth, the depth below the surface and the thickness of the water-bearing stratum were determined. The tube was then cleaned, filled with water to hold back the sand, and a strainer of proper length inserted and held in position by means of wooden rods. The pipe was next pulled back until the openings on the strainer were exposed to the water-bearing stratum. A swedge block was finally used to fasten the lead rim or joint on top of the strainer to the well pipe. The strainers in the push wells were 24 and 42 feet long respectively,

while those in the vertical wells were from 8 to 17 feet. After they were swedged into the pipe, and the well thoroughly pumped and washed by steam, the water was perfectly clear and free from sand.

The next set of wells were for test purposes, and were all 2 inches in diameter. Some of them were merely open pipes down which streams of water were forced by a pump, thus forcing passages into which the pipes dropped of their own weight or under light blows. The remaining wells were driven without washing, and did not give as good results as those sunk with water.

Another set of wells was driven by a still different plan. A pipe was started in the ground by fastening a heavy clamp about it firmly, and striking this clamp with a hammer sliding on the pipe and furnished with four handles, by which it could be lifted by several men. As the sinking progressed the clamps would be shifted higher, and finally the pipe was firm enough to allow a platform to be attached to it about 6 feet from the ground. The men stood on this platform and their weight aided the driving. Moreover, it was unnecessary to shift the clamp so often with this arrangement as where the men stood on the ground.

AIR IN WELLS.

The greatest difficulty in operating a driven-well plant is due to accumulations of air. Some of this air enters the wells with the water, but most of it probably leaks through the joints of the suction and branch mains. A line of pipe which is perfectly tight under hydraulic pressure may nevertheless leak when tested with air. Consequently the greatest care should be taken to have the entire system of wells and pipes as tight as it can be made. It is hardly less imporant to have as few bends in the pipes as possible, and to have the latter so large that the water will flow through them at a low velocity and with correspondingly slight friction. Everything should be done to reduce to a minimum the resistance encountered by the water in passing from the ground to the pumps. Some engineers believe that it is advisable on this account to connect the wells with the suction mains by diagonal branches, which deflect the course of the water some 45 degrees only, instead of 90 degrees as in the usual method of connection. The author has heard less complaint about air in plants with small suction pipes running down inside the main well tube

than in those where the well consists of a single tube, which is itself connected to the system of suction pipes.

The construction of the suction pipe is very important. It should consist of cast-iron flange pipe, with the flanges carefully trued. In one exceptionally tight main each joint was made with a copper gasket placed between a pair of rubber gaskets. Joints are also made with lead rings wrapped about with lamp cotton, or covered with cloth saturated with tar; in fact, there are many ways of making them, although that first mentioned is perhaps the best. The branches which have screw couplings should have the threads cut with more than usual care to secure tightness, and if red or white lead is employed, it should be used sparingly. Each well should have a gate on the branch leading to it, and care must be taken that the gate stem is tight. A patented joint by which a connection is made air-tight by forcing lead against the screw threads by means of a set screw seems particularly useful on the screw-joints of a driven-well plant. All pipes should be laid carefully to line and grade. If there is any doubt as to the stability of the bed on which they rest, they should be supported on piles or cradles which will ensure safety from all settlement. The grade or drop toward the wells should be not less than 6 inches in 100 feet, and there must be no summits or bends of any sort in which air can collect. Another point to be taken into consideration is the protection of the mains from the sun. If this is not done in some manner, the expansion of the main suction pipe in summer may be the cause of troublesome leaks in some of the small joints toward the ends of the line.

The mains end in air separators, which are chambers where the air is liberated from the water and removed through pipes running to wet vacuum pumps. There are many forms of these separators, each contractor having his favorite. He is usually decidedly averse to explaining its details, but willing to install it under good guarantees that it will work satisfactorily. It is not surprising, therefore, that little information concerning them has thus far been published. Many are automatic in action and actuated by a float in the upper part of the separator. As the air collects the float naturally settles with the depression of the water level in the chamber. When it has fallen a certain amount it opens the throttle valve of the vacuum pump in one way or another. If the air comes in faster than the pump removes it, the

float sinks farther, and adjusts the throttle so as to give the pump a greater speed. When the air is removed the float rises, and closes the throttle in so doing.

Many separators have no such automatic features. Some of them are merely large chambers with a smaller chamber on top provided with a gauge glass by which the amount of air within can be readily determined. The engineer in charge fixes the speed of the vacuum pump by observing the gauge, and, as the amount of air carried by the water from the same battery of wells does not vary very much, such separators answer fairly well.

A separator used by William D. Andrews & Brother on driven-well plants built by them is probably among the simplest that can be employed, and possesses the additional merit of having worked for a number of years with complete satisfaction to the engineers in charge of the plants. It depends for its action upon the fact that water cannot be raised by suction more than about 34 feet theoretically; practically, several feet less. It is a large vertical, cylindrical, sheet-iron tank or chamber on top of which is a much smaller one, perhaps 6 inches high, and rising from the top of the latter is a pipe about 3 inches in diameter. This runs to an elevation of about 35 feet above the level of the upper pump valves. The air collects in this pipe, which is closed at the top, and is removed through a smaller pipe running up within it nearly to its top. The inner pipe is connected with the air pump, and on account of the height at which air enters it, comparatively little if any water is mingled with the air. The outside vertical pipe has to be guyed securely and does not present a particularly attractive appearance, but these are the only objections against this plan, so far as the author has been able to learn.

The air separator used on the first permanent driven-well plant at Lowell, Mass., was designed by Mr. B. O. Gage, and is described substantially as follows in a report by Mr. George Bowers, City Engineer of that place. It is on the suction main just outside the pumping station, is circular in plan and made of boiler iron. It is dome-shaped, 28 inches high at the side, 44 inches in the center, and contains three dams. The water coming from the wells passes over the first dam, under the second, over the third and thence into the pipe running to the pump. A 6x8.5x6-inch wet vacuum duplex pump was connected with the top of the separator to take away the air liberated as the water

flowed over the dams, but it proved too small and a 7.5x10.25x10-inch pump was put to work with the first. After a two-months' trial of the separator, the middle dam was removed, with good results. As the plant is composed of 5,806 feet of suction pipe and branches, there is a great number of joints to be kept tight, and it requires constant care. A man goes over the line every morning with a pail of asphaltum paint, and if there are any leaks he repairs them.

WELL SPECIFICATIONS.

Through the courtesy of Mr. Freeman C. Coffin, M. Am. Soc. C. E., the writer is enabled to close his portion of the discussion of underground supplies with a number of extracts from the specifications of the Wellesley Water-Works new well plant, which was designed by Mr. Coffin:

"Wells.—The wells are to be of 2½-inch wrought-iron pipe; this pipe will be furnished in approximately 5-foot lengths, with screw threads cut on both ends and a drive-well coupling furnished with each piece. The piece for the lower end of the well, or point, will be furnished ready for driving. In driving the wells the washing out process by water pressure shall be employed. The contractor shall provide a suitable plant for this purpose. Men skilled in this class of work shall be employed. Whenever the engineer shall require it, a diaphragm pump shall be attached to the top of the well and a test made of its capacity. Each well shall be driven to a depth where the most satisfactory flow of water can be obtained. No well will be connected with a depth of less than 28 feet from the surface of the ground to the top of the holes in the point. Any well driven by the orders of the engineer, if abandoned, will be paid for at the regular rate, and the contractor will be required to remove the pipe at his own expense, care being taken not to injure it by such removal.

"The joint will be made with the material (elastic cement or otherwise) furnished by the committee.

"After each well is driven to the proper depth it shall be pumped from at its full capacity until no sand or gravel rises with the water.

"Well Connections.—The contractor shall do all trenching and back-filling and furnish all necessary lumber for sheeting and bracing at his own expense. The top of each well shall be cut off to receive the long-bend T-branch, 6 inches lower than the center

of the main to which it is to be connected. A screw thread shall be cut on the top of the pipe 1½ inches long; this thread shall be made with great care, and shall make an air-tight joint with the T-branch. The contractor shall be required to cut a similar thread upon one end of the short horizontal piece of pipe which connects the T-branch to the gate. The connection between the well and the main will be finally closed by making the joint between the companion flanges on the 2½-inch gate. All of the screw and flange joints on this connection shall be perfectly air-tight.

"Suction Mains.—The contractor shall do all the necessary trenching and back-filling, shall furnish all the lumber required for sheeting and bracing, and for the permanent platform and blocking under the pipe, as shown on the detail drawings. The lumber shall be good sound spruce acceptable to the engineer. He shall lay the pipe to the grades given by the engineer approximately as shown on the profile. All of the joints in these lines shall be run with pure lead and carefully calked. The lead in the joints of the 12 and 16-inch pipe shall be 2½ inches deep, and in the 6 and 8-inch pipe 2 inches deep. All joints must be air-tight. In making these joints the water in the trench must be pumped down to the bottom of the platform and kept there until they are all calked.

"Running Joints.—Before running the lead the joints shall be carefully wiped out to make them clean and dry; the joint shall be run full at one pouring, and the melting pot shall always be kept within 50 feet of the joint about to be poured.

"Extra Foundations.—If in laying these mains the foundation in any part of the trench should be found to be unsuitable in the opinion of the engineer, the contractor shall put in such foundation as the engineer may direct, and for such and all other extra work shall be entitled to be paid the actual cost thereof, and 15 per cent. additional for contractor's profit, use of tools and general superintendence.

"Tests.—Each section of the suction main and of the well connections as far as the 2½-inch gate shall be tested in the following manner:

"The section to be tested shall be shut off by the gate at each end; an air-pump shall be connected with the main. The trench shall be filled with clean water, either by infiltration from the

ground or directly from the brook. An air pressure of 50 pounds per square inch shall be applied to the mains and maintained as long as the engineer may direct. If this pressure can be maintained without pumping, the work will be considered tight and satisfactory. If it is necessary to run the pump to maintain the pressure, search shall be made for the leakage, which must be found and repaired in a manner satisfactory to the engineer. The contractor shall furnish all men, tools and materials to make these tests at his own expense. They shall be made under the direction of the engineer, and continued until he is satisfied of the tightness of the work. No back-filling of trenches or removal of sheeting or bracing will be allowed until each section is tested and orders to that effect are given by the engineer.

" Work to be Kept in Repair.—The contractor shall keep the work in good repair for the term of six months after the date that the water is let into the piping for the purposes contemplated in building said works, and shall correct and repair promptly during all that time all the leaks and failures of whatever description, and all settlements and irregularities of surfaces of streets and lines, the work in all respects to be in good condition at the end of that time.

"Wrought-iron Pipe.—The pipe shall be the best quality of American wrought-iron lap-welded pipe of the standard weight (or extra heavy, according to circumstances). Each length as it comes from the mill shall be cut into four pieces of approximately 5 feet each. The cutting shall be done by machine in such a manner that the ends of the pipe shall be smooth, true, and at right angles with the axis of the pipe. Each piece of pipe shall be straight and true throughout its length. A standard 2½-inch iron pipe screw shall be cut on each end of each pipe; these screws shall be cut by machine, shall be interchangeable, and shall so perfectly fit the taper and thread of the driven well coupling that the ends of the two pieces shall screw up close to each other in the centre of the coupling, and at the same time make a perfectly tight fit in the thread. This thread must be cut true with the pipe, so that when two pieces are screwed into one coupling their axes shall be in the same straight line.

"Well Points.—These points shall be approximately five feet in length. A screw thread shall be cut upon one end the same as on the ordinary pipe lengths specified above. There will be 48

holes drilled and tapped in the other end of the pipe and bushed with one-fourth-inch brass pipe, iron pipe size, as shown in the detail drawings. This pipe to be threaded and screwed into the holes in the iron pipe, cut off with about one-sixteenth-inch projection from either wall of the pipe and neatly headed down with a light hammer. A swage of a form satisfactory to the engineer shall be used on the inside for heading the bushing.

"The 2½-inch pipe for this point shall be the best quality American wrought-iron lap-welded pipe, extra heavy.

"Couplings.—The couplings shall be of the drive-well pattern, so called, similar to the sample in the office of the engineer. They shall have a sound, whole, clean-cut thread of 2½-inch iron pipe standard. The ends shall be reamed out about three-eighths-inch deep, large enough to just receive the outside of the unthreaded pipe. The total length of the coupling shall be 3½ inches.

"Testing Gates.—All of this work (gates) is to be used under a vacuum and careful tests must be made of each gate. These tests must be made by closing the valves to their seats, connecting an air pump or air pressure to the body of the gates and immersing the entire gate in clear water. One hundred pounds air pressure shall be applied. The gates shall sustain this pressure without showing a sign of air leakage. The valves shall then be opened wide and closed three times and the pressure applied again to discover if the movement of the stem causes leakage in the stuffing box.

"The 2½-inch gates and flanges shall be tested by screwing plugs into both ends and applying the pressure, both with the valve open and shut, the gate to be entirely under water at the time; the companion flange to be connected at time of test.

"If leakage is discovered in any part of the gates, it shall be optional with the engineer whether to have it corrected or to reject the gate."

CHAPTER XI.—DEEP AND ARTESIAN WELLS.

The geological features of deep and artesian wells have been fully described in a paper by Prof. Thomas C. Chamberlain, entitled "The Requisite and Qualifying Conditions of Artesian Wells," published in the Fifth Annual Report of the United States Geological Survey. This paper is the most complete presentation of the subject in English with which the author is familiar, and most of it applies equally well to non-flowing deep wells. As these reports can now be consulted in most public libraries, no attempt will be made here to present more than a mere outline of this phase of the subject.

The only way in which water can pass in any marked quantity through close-textured rocks is by means of fissures or channels formed by solution. Such rocks are those of the crystalline, limestone and clay classes, such as granite, greenstone, hard limestone and shales. While they may be full of cracks and crevices on weathered surfaces, they are generally homogeneous at moderate depths, and there is little likelihood of securing water from them. Limestones near the surface are often hollowed out into large passages by the dissolving action of water passing through them, but this action is chiefly found where the limestone is not overlain by other rocks. Limestones that were once channeled in this manner and afterward covered with a thick mantle of clay have been found to yield a good supply, but, as a rule, fine-grained limestone is a poor rock in which to look for much water of a quality fit for domestic and boiler uses. Beds of sand, gravel, sandstone, conglomerate, porous chalk and coarse, granular limestone are the rocks in which water is generally found. Naturally the best confining strata are those least pervious to water, such as clay, clay shale, shale limestone, shale sandstones and the various crystalline rocks.

The upper and lower confining strata are not of equal importance, and fissures may exist without detriment in the stratum be-

low the porous bed which would have serious consequences in that above. Fissures in the lower confining bed will rarely cause trouble unless they open into a porous stratum still lower, which has an outcrop at a lower elevation than the top of the well. In such a case the water will escape through this outcrop and will not flow out of the well, which will become of the non-flowing type in spite of the fact that its top may be much below the outcrop or gathering ground of the main porous bed.

There is one feature to be observed in connection with the upper confining stratum which modifies in a measure the effect of permeability in this bed. If the stratum is overlain with material in which the natural ground-water surface is at about the same elevation as the collecting area of the water-bearing stratum to be tapped by the well, the upper confining stratum may be somewhat permeable and yet not permit the passage of the artesian water owing to the hydrostatic pressure exerted on it from above. The lower the ground-water surface is below the elevation of the outcrop of the porous stratum, the greater is the effect of permeability in the upper confining stratum.

There is always some chance of failure in attempting to secure an artesian supply from a water-bearing stratum which outcrops not far from the well at a much lower elevation than the surface where the well is sunk. Nevertheless, the frictional resistance of the stratum to the flow of water from the site of the well to the lower outcrop may be so great that the water will rise to the top of the well. "Several important wells at Oshkosh, Fond du Lac, Watertown and Palmyra, Wis., flow from formations that outcrop within 50 miles at notably lower levels. These outcrops, however, are not in the line of slope from fountain head to well, but more nearly along the line of strike at right angles to it. All these wells probably owe their success to the high subterranean water level between the wells and their sources, but resistance to flow through the water-bearing bed seems also to serve an important function, unless the entire head would be relieved through the low outcrops."

When the topographical and geological conditions have been determined, after careful consideration, to be favorable for the construction of a deep or artesian well, it is necessary to ascertain how many wells in the vicinity reach to the stratum it is proposed to tap, and how they affect one another. If the starting

of a well causes much diminution in the discharge of others in the neighborhood it is an indication that about all the available water in the porous stratum is already being drawn from it. Another well may so diminish the flow from those previously constructed that serious inconvenience may arise. If, however, the starting of one well has little or no influence on those about it, one more may be sunk with fair assurance that it will not be a source of vexation.

It is, of course, desirable to sink the well some distance into the porous stratum so that, by perforations in the lower portion of the pipe, the water may enter the well through a much larger surface than is offered by the cross-section of the pipe itself. In case the flow remains inadequate after the well has been sunk far into the water-bearing stratum, various expedients may be tried to increase it. One of these is the explosion of a torpedo at the bottom of the well. This rends and shatters the rock, and opens fissures through which the water may pass more freely to the well. It is a common practice in the Pennsylvania oil regions, but has not been adopted very often in developing water supplies, although equally well adapted to such work.

The usual method of increasing the flow of a well is to ream it to a larger diameter. This is more expensive than shooting it with a torpedo, but it has advantages worth noticing, as the first bore need be only a small one, easy to drill and involving a comparatively small loss if it has to be abandoned from failure to strike water. "From the character of the flow obtained by the first operation it is possible to anticipate what will be the probable result of the enlargement. If the water issues with great force it is manifest that the larger bore will greatly increase the delivery, because, in addition to the increased size, the friction is relatively less. If the flow be gentle and the head known to be high, it is clear that the conveying stratum must interpose obstacles, and the indications are unfavorable to a very great increase from the enlarged well. If the fountain-head is low, a full, gentle flow is the natural sign of a generous stream, which might give an almost equally flush discharge from the enlarged bore."

These considerations also govern the choice between one large and several small wells. If it is known that the water-bearing stratum will afford a large supply, then a large well will probably be satisfactory. If there is any uncertainty as to the character

of the stratum, it will probably be desirable to use a number of small wells, driven far enough apart to tap an extensive area of water-bearing rock. If these are shot with torpedoes it is evident that they will draw water from a far greater portion of the stratum than a single well of any reasonable diameter. If a number of wells are employed it is preferable to locate them on a line at right angles to the direction in which the water flows in the porous stratum, or along a line somewhat convex toward the direction of the flow. The latter plan is probably the better, as it tends to increase the discharge from the wells on the ends of the line.

SINKING WELLS.

The methods of sinking wells to considerable depths may be classed under percussion, diamond and rotary drilling.

The percussion drilling is merely an elaboration of hand drilling and in its simplest form, using a spring pole, it has been employed for many centuries. Although drilling machinery is now supplied at low cost by a number of manufacturers, the spring pole nevertheless may prove useful in sinking wells to depths of 200 to 250 feet. The following description of the method of using such an outfit presupposes some knowledge of artesian well sinking, which is readily acquired from Trautwine's "Pocket Book" or the catalogue of any manufacturer of well-sinking apparatus.

The spring pole is generally a sound, round pine pole about 30 feet long, 10 inches in diameter at the butt and 6 inches at the tip. It is firmly embedded in the ground in an inclined position, with its free end over the spot chosen for the well. It is usual to sink a shaft a number of feet from the surface, because it is cheaper to do this than to drive a well the same number of feet at the end of the work, and also because a pit below the platform on the surface enables the operations to be conducted more rapidly. Three inches is about the greatest diameter of wells sunk in this manner, and the method is accordingly useful only where small supplies are desired.

After the spring pole has been erected, a shaft excavated to a depth of, say, 8 feet, and a windlass provided on the platform at the top of the shaft, it is usually necessary to drive a casing pipe to the rock. This is done by means of a heavy iron-bound wooden hammer, about 4 feet long, 18 inches in diameter at the bot-

tom and 12 inches at the top, with handles for guiding it. A rope is attached to the end of the spring pole and the hammer is hung from it by a hitch easily loosened when it is desired to shift the position of the hammer. A section of pipe about 6 feet long is then provided with an annular shoe at the bottom and a solid cap at the top. It is guided by blocks and sunk as far as possible by the hammer, the spring pole lifting the latter and thus greatly diminishing the work to be done by the men. After driving 6 or 8 inches the knot by which the hammer is hung from the pole is loosened and the latter lowered to within an inch or so of the cap. In this way little strength is expended uselessly in overcoming the spring of the pole. When the top of the pipe has been sunk to within 6 inches of the surface, the cap is removed, the thread of the pipe oiled and another length screwed to it. This length is forced down in the same manner as the first, and the process repeated until rock is reached. It is generally necessary to assist the driving every few feet by removing the soil within the pipe with an earth auger. This is screwed into the earth as far as it will go by means of a long shank to which pieces may be connected as the depth increases. When the auger is full it is pulled to the surface and its load dumped. The casing can be drawn out of the hole, if desired, by a clamp at its top against which a couple of jackscrews are operated.

After the pipe has reached rock a string of tools ending in a drill bit is lowered to within a few inches of the bottom and suspended from the spring pole. The string is then churned up and down, and the rock gradually pounded away on the line of the hole. It is generally necessary to ream the hole true after it has been driven with the bit; the reamer is operated in the same manner as the drill. The splinters of rock are removed by a sand bucket whenever it is apparent they are interfering with the action of the tools. Unless the bore hole contains water, enough must be poured into it to give a depth of several feet, or progress will be slow. If it is desired to continue the casing into the rock, it will be necessary to enlarge the bore hole with an expansion drill. About six feet a day is said to be a fair day's work in hard sandstone with such a rig. A full description of the tools required and the methods to be followed will be found in a paper by Mr. Edgar G. Tuttle in the "School of Mines Quarterly" for November, 1894.

The apparatus just described is the simplest that can be employed in sinking a deep well. It has been modified in many ways by replacing the pole by a system of levers or cams operated by horse or steam power. The best way to obtain information concerning the apparatus is to write for the catalogues of the various manufacturers whose business cards appear in the advertising columns of "The Engineering Record." Engineers having access to the publications of the Second Geological Survey of Pennsylvania will find a particularly good description of well-sinking in Mr. John F. Carl's report on "Oil Regions."

Diamond drilling is done by means of a tube having a boring head at the base, set with small pieces of diamond. Water is forced down the tube, which is pressed against the rock and rotated. In this manner a hole may be drilled to great depths, and, as the boring head leaves a core of rock, it is possible to obtain from it a very good idea of the character of the strata penetrated. Much the same plan has been adopted in some recently built machines for sinking wells in earth; these machines are known as the rotary or rotary-jetting type, and their use is indicated in the following paragraph.

The method of sinking the wells of the Natchez, Miss., waterworks was to put in a casing first to exclude the surface water, and then commence with pipe of the desired size and force it nearly through a 40-foot bed of water-bearing sand about 260 feet below the surface. Mr. P. K. Yates, M. Am. Soc. C. E., states that the pipe had a notched cutting edge and gradually wore its way down by being revolved by a suitable machine. Water was continually pumped down the center of the pipe, washing out all the material within and driving it to the surface on the outside of the pipe. Hard material was sometimes encountered, when drills would be dropped down the pipe and the obstruction drilled through. The pipe had to be pulled up occasionally to put on a new cutting edge; hydraulic jacks, connected with powerful pumps, were used for this purpose. After a pipe was in place, a strainer 20 feet long was dropped into it and the pipe pulled up that distance, the strainer remaining in the sand. A patented connection was then made to prevent any water entering the pipe except through the strainer.

SPECIFICATIONS.

Most wells are now sunk by contract; the following extracts

from the specifications of the Galveston Water-Works, prepared by Mr. Wynkoop Kiersted, M. Am. Soc. C. E., are here given as suggestive for such documents:

"The location of each and all of the wells must be submitted to the engineer and receive his approval before any well is bored. The depth of each well will be such as shall be determined by the engineer at the time of boring it.

"The plans for the conduit and for the lateral pipes and attachments will be prepared by the engineer as soon as the number and economic distribution of the wells are determined by means of tests hereafter specified.

"The casing to be used for the artesian wells shall be what is known as lap-welded drive pipe of the following minimum weights and thicknesses:

		Thickness.	Weight per foot.
6	inches inside diameter.....	$\frac{13}{32}$ inch	25 pounds.
8	" " "	" "	35 "
10	" " "	" "	44 "
12	" " "	" "	52 "
13	" " "	" "	58 "
14	" " "	$\frac{7}{16}$ "	67 "
15	" " "	" "	72 "

"The thickness of pipe metal shall be calipered. No pipe of less inside diameter than 6 inches to be used. The couplings shall be of the best wrought-iron, lap-welded, drive-pipe sleeve sockets. Both pipe and sockets shall be fitted with perfect V-shaped threads, not more than eight to an inch, and tapered so that the pipes shall butt against each other when screwed up. All inside burrs shall be removed. Every pipe shall be perfectly straight and round and free from blisters. The metal shall be of the best quality of wrought iron, suitable for perfect threads being cut thereon. In sinking the wells the openings between the pipe circumference caused by sinking a small pipe inside of a larger one shall be completely closed, so as to prevent the entrance of sand and water.

"Each well shall have a strainer of perforated brass, or of other equally as good non-corrosive material acceptable to the engineer, of such length as shall be best suited to the depth of the sand stratum. The clear strainer opening must have a combined area of not less than ten times the inside area of the strainer cross-section. The size of the openings shall be graded with respect to the coarseness of the sand in such a manner as to exclude sand

but freely permit the flow of water. The thickness of the metal at any point in the body of the strainer shall not be less than one-eighth inch, and otherwise the strainer shall be sufficiently stiff for the work. There shall be a metal plug in the bottom of the strainer completely excluding all sand. The strainer shall be attached to the casing by a water-tight joint.

"The lateral pipes leading from any well to the conduit shall be of cast iron with hub and spigot joints. The size of the pipe shall depend upon the flow of the well which it connects with the conduit and upon the distance from the well to the conduit, and as may be ordered by the engineer.

"The main conduit pipe shall be of cast iron with hub and spigot joints 36 in. inside diameter, laid on a level.

"The entire system of conduit and lateral pipes shall be tested when completed to a hydrostatic pressure of 100 pounds per square inch and the pressure maintained there until there is satisfactory evidence of tight joints. The contractor shall frunish pumps, tools and all facilities for this test.

"All material and workmanship shall be guaranteed for a period of six months from the completion of the work, and the pipe lines shall, during that period, be maintained by the contractor in a perfect and water-tight condition and the wells be flowing freely. There must be no evidence of sand accumulating in the wells as a direct result of imperfect joints and an imperfect plugging of the bottom of the strainer."

It will be seen from these specifications, that the conditions laid down for the contractor at the outset of such a piece of work cannot be very definite. The Galveston wells were of the simplest type and required no rock drilling, yet the specifications could give absolutely definite information on very few points. Specifications for wells in rock would necessarily be different from those quoted, but it will probably be found best in draughting them to avoid giving any particulars which are not absolutely certain, and refer the contractor to the engineer for directions as the work progresses. Many of the minor features of a deep well plant cannot be definitely determined until the wells are sunk and tested, and to give unmatured details in the original specifications will furnish the contractor a hook on which to hang troublesome claims for extras, a source of annoyance and often of bad feeling.

Wells are now cased to their full depth, with few exceptions. Sometimes the casing is in several sizes, so that the well resembles a drawn-out telescope; if this plan is adopted the connection between pipes of different diameters must be water-tight. More frequently, however, the pipe remains the same diameter from top to bottom. Which plan will ultimately prove the more satisfactory cannot always be foretold, as conditions may render impracticable the sinking of a pipe of uniform diameter.

When the casing is not carried down into the stratum it is desired to tap, the water may be contaminated by that from higher strata. In this case the pure water must be drawn off through a small tube sunk for the special purpose, which is provided with a packing on the outside for hermetically sealing the good water from that above. A number of devices employing rubber rings or helices are made for the purpose, while if none of them is at hand a seed bag will probably answer. To make such a bag, an open cylinder or bag of stout leather is slipped over the inner or discharge pipe, and tied firmly at the bottom. The space between the bag and the pipe is then filled with dry flaxseed and the upper end of the bag lashed firmly to the pipe. This bag is lowered until it reaches the proper depth to intercept the flow of undesirable water toward the bottom stratum, and held there until the water has caused the seed to swell and fill tightly the space between the bore-hole and the discharge pipe.

YIELD OF WELLS.

The development of deep well supplies, either artesian or non-flowing, is a matter requiring careful study, for the subject is by no means as simple as it seems. The elements of the problem are all that can be discussed in these articles. In the first place, it is necessary to refer again to the geological reason for wells, shown diagrammatically in Figure 27. The well in this figure is fed by the rainfall on an outcrop of porous material much higher than the surface of the ground where the well is sunk. The consequence is that the frictional resistance to the passage of the water through the stratum is less than the difference in elevation between the outcrop and the valley, and the well is of the flowing type. Suppose it is prolonged upward by a series of pipes, as was done in some interesting experiments by Mr. T. T. Johnson at Savannah, until the elevation is reached at which no water is discharged. This is marked 0 in the diagram. Now let these

pipes be dropped 25 and 50 feet respectively, and the discharge measured at each elevation. These discharges will be, say, 50,000 and 100,000 gallons in 24 hours, as marked on the diagram. The reason for these different rates of discharge is manifestly that the head between the catchment area where the rainfall is collected and the elevation at which the well discharges has been varied, the greater the head the greater being the discharge. The frictional resistance is practically, although not exactly, the same whether the discharge is at the 0, 25 or 50 elevation.

Continuing the experiment by measuring the flow when the well is stopped below the surface of the ground at elevation 75, so that the water does not have to rise above that elevation, let it be assumed that the measured discharge is 150,000 gallons per day. It will be noticed that if the discharges at the various elevations are represented by the lengths of horizontal lines, as shown in the

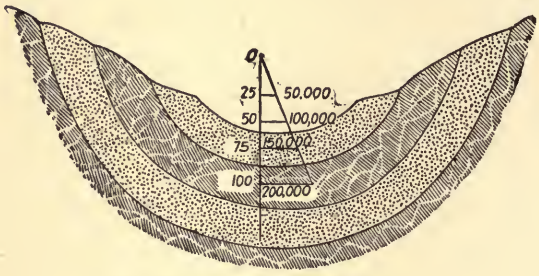


FIGURE 27.—YIELD OF WELLS.

illustration, the line connecting the extremities of these horizontal lines is straight. While this is a theoretical case, it is fortunately true that these discharge curves, as they are called, are usually found by actual experiment to be practically straight lines.

In case a well has been driven, the discharge should be measured when the outlet is at several different elevations. This is an easy matter when the well is artesian, but somewhat more difficult when it is non-flowing. In the latter case, the discharge at various depths below the surface may be found by using horizontal plunger pumps and varying the suction lift from time to time. That is to say, the pump can be run at first with a suction lift of 5 feet. After measuring this discharge with some care, the pump can be speeded until the suction lift is 10 feet, and the process

continued until the lift is 25 feet. Such a procedure, which is equivalent to cutting off the well at these different depths, so as to let it discharge freely, will enable the discharge curve to be drawn with sufficient accuracy for all practical purposes. It may happen, of course, that the water will not rise in the well high enough for a pump on the surface to operate properly. In this case, some of the various pumping apparatus referred to in the chapter on pumping will be necessary, the air lift being particularly useful on account of the small obstruction it offers to the discharge of the well.

In studying these discharge curves it is necessary to bear in mind that while the yield of a well increases uniformly as the point of delivery is lowered, a depth may be reached at which the quantity drawn off is greater than the supply. Hence such a heavy draught will gradually diminish the supply, and to avoid any such occurrence after works have been built, the wells have sometimes been pumped at the maximum delivery desired for a period of several weeks or months before going ahead with the construction of the plant.

The great advantage of knowing the discharge curve of a well is the fact that it enables the engineer to select his pumping apparatus intelligently. He may find it desirable, as at Memphis, Tenn., and Riverside, Ill., to place a horizontal plunger pump many feet below the surface, or the diagram may be such as to convince him that similar pumps on or slightly below the surface will furnish an ample amount of water. The choice between deep well pumps and air lifts may also be aided to a certain extent by the character of the discharge curve.

In testing the capacity of any well care should be taken that none of the water escapes into overlying strata before reaching the surface, or that the volume drawn from the well does not include impure water from strata which must be subsequently shut off. A possible case also is that of a bore hole which has been sunk through several strata bearing good water in the vain hope of finding larger volumes lower. In case the whole is not plugged just below the lowest water bearing stratum it will afford a path for serious waste in case any of the lower, dry strata are permeable.

The history of the development of the water supply at Rockford, Ill., affords a good example of the possibilities of deep wells.

Under that city are the two well-known sandstone formations from which many supplies in Illinois and Wisconsin are drawn. The lowest formation is the Potsdam. Above that is a stratum of impervious magnesian limestone a little over 100 feet thick, and then comes the water-bearing St. Peter sandstone, about 200 feet thick. This in turn is covered with glacial drift. The first deep well supply was from five 8-inch wells sunk from 1,300 to 2,000 feet to the Potsdam sandstone. After a short time the yield of these wells proved insufficient, and others were sunk into the St. Peter sandstone. Even these proved unable to supply the demand, and an air lift system was introduced, as described in "The Engineering Record." This furnished the needed increase in the volume of water, but it required too much coal to be satisfactory to the authorities. So a careful study of the wells was made in much the manner just outlined. The air lift was used to determine the discharge, the elevation of the water in the wells being determined by a float attached to a steel tape. It was finally decided that the best way of securing the desired quantity of water was to sink a shaft and connect it with the wells by tunnels. Three centrifugal pumps have been placed in alcoves at the bottom of the shaft, and are driven by rope transmission from the three independent vertical engines at the surface. Each pump is of 3,000,000 gallons capacity.

Before closing this necessarily brief discussion of deep wells, the writer feels compelled to point out again that the development of a supply from a source several hundred feet below the surface is naturally somewhat of a lottery. This is shown by the experience in many places, although but one instance can be cited here, that of Chester, S. C. The city first engaged a geologist to examine the locality; after two days' study he selected three points where artesian water would probably be found at a depth of 400 to 500 feet. Mr. Perry Andrews, of Elmira, N. Y., was engaged to sink the well. He began boring in a hollow near a stream, and at the end of five or six months had sunk a bore hole about 500 feet into the rock. A supply of 30,000 gallons a day was obtained, but, on Mr. Andrews' advice, the site was abandoned and work begun a mile away on a hill side. This attempt was successful, for a supply of about 250,000 gallons was obtained by sinking a well 400 feet deep.

QUALITY OF GROUND WATER.

A few words should be added at this place concerning the quality of well waters. Where the wells are shallow they are exposed to contamination from the surface, and, as a matter of fact, shallow wells in the vicinity of dwellings are always regarded with suspicion by sanitarians. In the interpretation of an analysis of water from such a source it must be remembered, however, that the soil about a well may contain a certain amount of harmless organic matter which will cause the amount of nitrates in the water to be unusually high. In the preliminary work connected with a chemical survey of the shallow well waters in Illinois, Prof. A. W. Palmer adopted the following maximum limits of impurities, in parts per million, for supplies which may be used with safety: Total solids, 500; oxygen consumed, 2; chlorine, 15; nitrogen as free ammonia, 0.02; nitrogen as albuminoid ammonia, 0.05; nitrogen as nitrites, 0.001; nitrogen as nitrates, 15. The limit for nitrates seems very high if judged by the limits of 0.5 part in Maryland, 1 part in Iowa, or 0.9 part in Michigan, but "the fertile drift soils of Illinois are naturally highly nitrogenous, and it is probable that for this region the quantities of nitrates normally contained in unpolluted ground waters may range much higher than elsewhere." Even when Prof. Palmer applied the liberal limits mentioned, he found but few instances of shallow wells which could be used safely. In Massachusetts, the unpolluted ground water from near the surface is stated to differ from the unpolluted surface water in having only about one-tenth as much free and albuminoid ammonia with five times as much nitrogen as nitrates. These few observations will show how undesirable it is for any one not an experienced chemist to attempt to interpret the results of an analysis, and also how important it is that all the available information concerning the surroundings of wells and the strata they penetrate should be sent to a chemist retained to pass upon the potability of any supply.

As regards the Illinois wells, Prof. Palmer has made some interesting comments in his preliminary report, which are reproduced here on account of their somewhat general application:

"The waters from shallow wells are well aerated, and are clear, sparkling, cool, and of agreeable taste; those from the deeper (drift) wells, on the other hand, contain little or no oxygen, possess in many cases a disagreeable taste due to the presence of

marsh gas, accompanied occasionally by minute quantities of sulphuretted hydrogen, and are either turbid or become turbid quickly on exposure to air, owing to the oxidation of the iron carbonate which they contain and the consequent precipitation of the insoluble ferric compounds. The precipitating particles are often so minute as to be at first indistinguishable except from the color they impart to the water, but after a short exposure to the air the water becomes opalescent, then decidedly turbid; finally a brown deposit similar to iron rust is produced, and after this has separated the water becomes clear and colorless.

"Although these unpleasant characteristics of the deep drift waters give rise to much prejudice and objection to their general use for drinking, nevertheless, from the sanitary standpoint, they are usually to be preferred to the clear and palatable waters of the shallow wells, since the evidence of numerous analyses shows that they are far less subject to pollution with refuse animal matters than are the latter, while the organic matters which they contain are derived from the buried vegetable remains referred to above, and are comparatively harmless."

One source of dissatisfaction with certain ground-water supplies from wells or filter galleries near natural bodies of water is caused by the organism Crenothrix. These supplies are obtained by imperfect filtration from the neighboring stream or pond, and contain an unusual amount of free ammonia and some protoxide of iron. The bacterium has the peculiar property of separating the dissolved iron from water and incorporating it in its sheath, where it exists as iron rust. Such waters are unsatisfactory for use in the laundry, because white clothes become much discolored by the rust, and some supplies have been abandoned entirely on account of the trouble caused by this organism.

The most common objection to deep-well waters is their hardness, although many of them do not have this drawback. A valuable monograph on the subject, written by Mrs. Ellen H. Richards, was published in the twenty-seventh annual report of the Massachusetts State Board of Health. As that article may be too technical in its chemical portions for general readers, the following discussion of the subject is reprinted from a report on the water supply of Winnipeg, by Mr. Rudolph Hering, M. Am. Soc. C. E.:

There are two kinds of hardness: Temporary and permanent.

The former is usually caused by the carbonates and the latter by the sulphates of lime or magnesia.

Temporary hardness can be removed:

First.—By a sufficient quantity of soap.

Second.—By carbonate of soda (washing soda). The carbonate of soda unites with the bicarbonate of lime dissolved in the water, resulting in the formation of bicarbonate of soda and carbonate of lime. The former remains in solution and does not harden the water; the latter is precipitated as a fine, white powder.

Third.—By boiling. The bicarbonate of lime is decomposed by heat into carbonic acid, which escapes, and carbonate of lime, which is precipitated as a fine, white powder.

Fourth.—By a solution of freshly burnt lime, or lime-water. The carbonates of lime and magnesia are changed into mono-carbonates by the hydrate of lime uniting with the extra carbonic acid, which is either free or combined as bicarbonate in the hard water. The resulting insoluble mono-carbonates deposit as a fine powder. Carbonate of lime is not entirely insoluble in water, and a small portion always remains in it. The soluble bicarbonates of lime or magnesia, having thus lost half their carbonic acid, are reduced to the same insoluble mono-carbonates and are also precipitated. This process, being the least expensive, is the one here recommended.

Permanent hardness can be removed:

First.—By a sufficient quantity of soap, as before.

Second.—By carbonate of soda. The soda in this case unites with the sulphate of lime or magnesia dissolved in the water, resulting in the formation of the neutral and inert sulphate of soda, and the insoluble carbonate of lime or magnesia. The former remains in solution and does not harden the water; the latter is precipitated as a fine white powder. In cool water the presence of free carbonic acid, or of bicarbonates, interferes somewhat with this reaction; but the combined lime-and-soda process obviates this difficulty to a large extent. As permanent hardness is usually present with temporary hardness, the lime and soda can be mixed and together added to the water.

To remove permanent hardness this process is the least expensive one for city supplies.

CHAPTER XII.—PUMPS.

The pumping plant used in works of the size considered in this series of articles may be roughly divided into two classes, well and low-lift plants. Under the first head is included the apparatus used to raise water from wells in which the power or driving machinery cannot be placed. It includes deep well pumps, air lifts and a number of special appliances, which have been used in a few instances. Under the second head are included the plants which are placed wholly on or near the surface. This is merely an arbitrary classification to simplify the arrangement of the information on the subject presented in this chapter and the next, which are intended solely to assist water-works committees and others, not acquainted with engineering work, in selecting pumping machinery.

The pumps used on or near the surface may be employed to force water into a reservoir of some sort, which keeps the head constant against which they work, or the machinery may force water directly into the pipes and be designed to furnish higher pressures than usual when fires or other emergencies occur. The latter system of operation was formerly known as the Holly, but is generally spoken of to-day as direct pumping.

Pumping machinery is also classified according to the method by which it is driven, whether by a direct connected steam end or by power furnished in some other manner. The greatest proportion of the pumping plants in this country are operated by steam, and it is therefore natural to turn to them first; it is well to remember, however, that gasoline engines and electric motors have only recently come into favor for driving pumps, and on account of their advent the next ten years are probably destined to see a considerable change in small pumping plants.

The best method for any one interested in the purchase of pumping machinery, but uninformed as to its general construction, to obtain a general knowledge of the subject is to secure the

catalogues of the manufacturers whose cards appear in "The Engineering Record." These firms have invested a large capital in their plants and have taken advantage of every improvement that specially trained mechanical experts and ample financial resources could find. American pumping machinery is to-day so admirable that hydraulic engineers very rarely make any plans for this portion of a water system, and content themselves with specifying the results they wish. Hence, no attempt will be made to explain more of the design and construction of pumps than is necessary to point out in a very general way the conditions for which each class is best adapted.

The larger part of the steam pumps in use in American water-works are of the direct-acting type, in which the piston rod of the steam cylinder is prolonged to form the rod of the piston or plunger in the corresponding water cylinder. Such a pump can be built for less money than a crank-pump of the same capacity. It occupies less space and weighs less. On the other hand, it is more wasteful of steam than the crank and flywheel pumps, unless fitted with special devices raising its price.

The simplest pumps are, of course, those with a single steam cylinder and a single water cylinder. Horizontal pumps of this type are sometimes used in water-works, mainly for feeding boilers and such purposes and in raising water from deep wells. The steam cylinder of well pumps is usually mounted on a standard which can be placed over the well hole, and the piston rod is continued downward by wooden bars to the plunger. Steam is admitted during the whole stroke and the breakage of the pump rods or the failure of the water supply is liable to wreck the cylinder, the piston offering no resistance under these conditions to the force of the steam. To provide against such an accident, it is usual to furnish the pumps with adjustable valves, which check the escape of the exhaust steam. These allow the exhaust to pass off with sufficient rapidity to produce little effect on the working of the engine, but in case the rod breaks or the water fails, allowing the piston to jump ahead violently, the exhaust is compressed into a cushion which checks the blow of the piston. Special arrangements are also provided for varying the amount of steam supplied to each end of the cylinders. The water end of these pumps will be referred to later. Some manufacturers supply a horizontal steam cylinder for well pumps, which is fur-

nished with a heavy working beam, like a large bell crank lever. One arm is pinned to a connecting rod running from the piston of the engine and the other is attached to the pump rods. Such an arrangement is preferred abroad to the vertical type usually installed in the United States, as it is unnecessary to move the steam cylinder or disconnect any of the steam or exhaust piping in order to reach the working parts in the well.

STEAM CONSUMPTION.

Except where deep wells are the source of supply, steam pumps are rarely used for water-works purposes without some attempt is made to use the steam expansively, and thus gain more work from it than is possible when full boiler pressure is exerted on the piston during its entire stroke. This advantage is secured by means of a second or low-pressure cylinder, placed in a line with the first or high-pressure cylinder, the pump plunger and the pistons in both steam cylinders being connected by one piston rod. The exhaust steam from the high-pressure cylinder passes to the low pressure cylinder and expands behind its piston, thus doing much useful work which is lost unless this expansion takes place. Owing to the peculiar features of direct-acting pumping machinery, it is impracticable to use steam expansively in a single cylinder steam end, in the manner followed in a flywheel engine having but one cylinder, and it is necessary to use a compound steam end to secure this advantage. In other words a compound steam pump is necessary to obtain the effect produced by a cut-off device on an ordinary engine for power purposes or on a crank and flywheel pump; it will save from 20 to 30 per cent. of the steam required to do the same work in a non-compound pump.

Still greater economy can be obtained by condensing the steam exhausted from the low-pressure cylinder, so that the motion of the piston is due to steam pressure on one face and a partial vacuum, due to the condensing apparatus, on the other face. The condensation is usually effected by playing fine jets of water through the exhaust steam and then pumping by an air pump the condensation and any air that may find its way into the condenser into the boilers or an adjoining waterway, according as the water is to be used over again or thrown away. The use of this condensing apparatus requires considerable water, but it saves about 20 per cent. of the steam needed when it is not used. In some localities there may be plenty of water available, but not

of a quality fit for boiler purposes; in this case a surface condenser may be used, which will return to the boilers all but about 5 per cent. of the water originally evaporated by them, the condensation being produced by the inferior but cheap supply. Sometimes the condenser is placed in the main suction or discharge pipe, where the condensation is effected by the water passing through the main pump. The air pump in both jet and surface condensers is sometimes driven by the main engine and is sometimes independent.

It should be mentioned here that it is customary to provide the compound pumps employed in direct-pumping water-works with special valves whereby steam at boiler pressure can be admitted to the low-pressure cylinders. In case of fire these valves are opened and the water pressure in the street mains raised considerably by the increased power obtained in this way.

Even the best of these compound pumps of the direct-acting type fail to use steam as economically as the crank and flywheel engines, and inventors worked for a number of years to provide an attachment which would remedy this disadvantage. Several arrangements have been designed and a number of them are in use. They are mechanical contrivances of much ingenuity, and fulfill their purpose satisfactorily. They are called high-duty attachments.

Another method of increasing the economy of a compound condensing pump is to add a third steam cylinder. In such a steam end the exhaust from the high-pressure cylinder is taken to the second or intermediate cylinder, and there made to do work while expanding, and is then taken to the low-pressure cylinder and made to furnish still more work by further expansion. Such a steam end is in most cases provided with condensing apparatus, and if it is also furnished with a high duty attachment, the pumping engine represents the most economical form of the direct-acting type now built.

A large proportion of the direct-acting engines now in use are of the duplex type, that is to say, they consist of two complete pumping engines placed side by side, and so connected that the steam valves are actuated, not by the engine they control, but by its mate. The effect of such a device, according to Mr. William M. Barr's book entitled "Pumping Machinery," "is to allow one piston to proceed to the end of the stroke, and gradually come to a

state of rest; during the latter part of this movement the opposite piston then moves forward in its stroke, and also gradually comes to a state of rest; but in its movement forward, and before reaching the end of its stroke, the slide valve controlling the first piston is reversed, and in consequence the first piston returns to its original position, and in nearing the end of its stroke, it reverses in a similar manner the slide-valve controlling the second piston; these movements are both uniform and continuous so long as steam is supplied to the piston."

The same author sums up the advantages of the duplex type as follows: "The very great success attending the introduction of the duplex pumping engine shows that it well provides the means of pumping heavy columns of water with ease and safety to the machinery employed, permitting the application of any amount of power required to lift the water column without violent or abrupt action upon the water, thus meeting an acknowledged demand that the rate of movement of the water column through the forcing main shall be as nearly as possible uniform, so that no considerable alteration of pressure shall be shown at any time while the pump is working. It also meets the requirement that the propulsion of the water shall be produced by the the use of the smallest practicable amount of moving material for transmitting the force of the steam to the column of water, in order to reduce to the lowest point the momentum of moving parts, and the hurtful effects due thereto in case of derangement of the valves or pipes. The time allowed at the end of each stroke before the piston takes up its return motion is sufficient to permit the water valves to seat quietly, and to allow the incoming supply to completely fill the water cylinder."

Crank and flywheel pumps differ from the direct-acting type in one feature, the result of their construction, which influences at times the design of the adjuncts to the force main. In direct-acting pumps the steam and water pistons move with practically the same velocity throughout each stroke, with the result that water passes through the pumps at a fairly uniform velocity. With crank pumps this is not true; the flywheel of such pumps revolves at a uniform rate, but this uniformity is maintained only by a variation in the speed of the steam pistons and hence of the water pistons. An important point about such pumps is the fact that the length of the stroke is definitely fixed; on this account

care must be taken with the steam piping to prevent any water entering in the cylinders. If this occurs something is apt to break, whereas in a direct-acting pump with its variable stroke, such an occurrence is not so liable to make trouble, and less pains need be taken with the piping in consequence.

The advantages of the type under consideration are stated as follows by Mr. H. P. M. Birkinbine: "A higher piston speed can be had with a crank and flywheel pump than if the pump were direct-acting, for the reason that in the latter type the termination of each stroke is defined and secured by steam acting as a cushion to counteract the force of the moving parts and of the water. In large steam pumps 100 feet per minute may be considered as the limit to safe piston speed. With pumping engines having cranks, connecting-rods and flywheels to terminate and define the stroke of the piston, any piston-speed possible to the pumps can be secured with safety. The power stored in the moving mass of the flywheel at the termination of the stroke, is carried to the beginning of the next stroke without any loss but that due to the friction of the moving parts and the resistance of the air to the motion of the flywheel. Then the practically uniform speed of the rim of the flywheel secures the desired motion for the piston through the connecting-rod and crank of the pump by gradually retarding the motion until the point of rest is reached, and accelerating it after the piston has passed that point." In brief, it may be said that the advantages of the crank pump are mainly in its greater economy in the use of steam and its higher piston speed as compared with the direct-acting pump. The former advantage means a decrease in fuel and in boiler capacity, the latter a greater volume of water delivered from water-ends of the same size.

Three factors should be considered in selecting a pumping engine from the various types mentioned, viz.: first, cost of engine and boilers; second, cost of operation; third, cost of maintenance and repairs, including the loss of time due to repairs. The second factor, the cost of operation, is influenced considerably by the length, diameter and nature of the force main, as will be explained later.

A moment's reflection will show that the first cost of a steam pump will depend largely on the steam end selected. The cheapest type, the simple non-condensing, direct-acting pump, will re-

quire three or four times as much steam and coal as a high class compound condensing high-duty pump and a much larger boiler capacity, for it cannot use steam at such high pressure or so economically. Mr. Charles A. Hague, M. Am. Soc. C. E., recently printed a table showing the effect of an increase in initial steam pressure on the duty which it is possible to obtain with compound and triple-expansion steam ends. A few figures from this table follow:

Duty per 1,000 Pounds of Steam.

Steam Pressure.	Compound.	Triple-expansion.
75	104,000,000 foot-pounds	——
80	106,000,000 "	——
85	107,000,000 "	——
90	108,000,000 "	——
95	109,000,000 "	——
100	110,000,000 "	125,000,000
105	111,000,000 "	126,000,000
110	112,000,000 "	126,000,000
115	113,000,000 "	127,000,000
120	114,000,000 "	128,000,000
125	115,000,000 "	129,000,000
129	——	130,000,000
133	——	131,000,000
137	——	132,000,000
141	——	134,000,000

It is important to bear in mind that the nature of the service an engine is called upon to perform makes a marked difference in the relative ultimate economy of the different types. A pump becomes more expensive as its type is raised, and in some instances the money saved by the use of one type will not equal its increased cost over a type of less economy, that is, one using more fuel to pump the same quantity of water. The matter of load will often have much weight in the choice of a pump; if the load is that due to a reservoir and is constant, triple-expansion will often be most economical where much water has to be handled, while with the direct-pumping system, where the load varies between night and day and runs up quickly during fires, a compound steam end may be preferable.

After the type of engine has been chosen it is an easy matter to determine whether it shall be vertical or horizontal. If the supply of water comes from a river subject to great variations in its surface level or from wells liable to become seriously taxed during times of heavy pumping, the pump end of the engine should be as low as possible. By using vertical engines this can be accom-

plished and still have the steam ends high enough to be readily accessible. Another advantage of the vertical engine is the comparatively small floor space it takes up. On the other hand, the horizontal engine in the smaller sizes costs about a fifth less, it is more easily examined and repaired, and requires no balancing apparatus to make it work smoothly on light loads.

POWER PUMPS.

Not many years ago power pumps were used in this country for water-works purposes only in connection with water-wheels. Plants of this type in some of the New England cities, Philadelphia, Richmond and a few other places were often described in early articles on water-works, and they still teach valuable lessons. But power pumping to-day is in a very different condition from what it was fifteen years ago, and no one interested in small water-works can afford to neglect it. The independent water end driven by a gas or gasoline engine, electric motor or steam engine is a combination which offers many advantages for small plants. It has already been mentioned that the small direct-acting steam pump is decidedly uneconomical in the use of steam. In Volume xvii. of the "Transactions" of the American Society of Mechanical Engineers there is a discussion on this subject from which a few statements may be taken with advantage to illustrate this lack of economy. Dr. Chas. E. Emery found in one large establishment many pumps using as much as 240 pounds of steam per net horse power per hour, and only in rare cases could one be found which was furnishing a horse power for each 80 pounds of steam; it was generally believed that about 150 pounds per horse power was the average consumption. These small steam pumps were replaced by power pumps operated by good high-pressure non-condensing engines. Mr. J. G. Winship stated that a large saving has been made by the Tidewater Oil Company by the use of power pumps driven by an automatic cut-off engine. But passing over the fact that small pumps do not use steam economically, power pumps offer a marked advantage in not absolutely requiring a steam engine and its attendant boiler plant and coal shed, for they can be driven equally well by a gas or gasoline engine or an electric motor. The gas engine is not used to any considerable extent in this country, but it has been so employed in many German cities, as is shown in an illustrated article in "The Engineering Record" of June 1, 1895.

Gasoline engines offer many advantages for small pumping plants, because of the ease with which their fuel can be transported, an important matter where the pumping station must be located at a place difficult of access, and because they require little attendance compared with a steam plant. The electric motor is as easy to look after, and where it is used there is no need whatever of transporting fuel. Hence it is not surprising that such sources of power are causing marked changes in the design of small pumping stations. Where a town operates a small electric lighting plant, and finds that it will be most advantageous to locate its pumping plant some distance from the built-up district, the use of electric motors driving power pumps should certainly be investigated. The lighting plant would run during the night and the pumping plant during the day, thus using the motor part of the station to the best advantage and saving part of the expense of installation and operation which must be met if independent lighting and pumping stations were constructed. There are good reasons for the belief that secondary pumping stations operated by electric motors will also be used to a considerable extent for high-service districts.

DETAILS OF THE WATER END.

The water end of a direct-acting or crank and fly-wheel pump served as the pattern for power pumps for many years, and where large volumes of water must be handled they are still employed. Such power pumps are generally driven by turbines through a train of powerful gears, and all parts are made heavier than is the case of similar parts of a direct-acting pump. In the latter the steam acts as a cushion to take up jars and shocks, but in the geared power pump there is a mass of non-compressible water in the turbines at one end of the mechanism and another mass of water in the pump cylinders at the other end, the two being connected by rigid machinery transmitting stresses instantly from one to the other and exposed to serious accident if not strong and well proportioned.

These water ends are made in the same manner as the ends of steam pumps, and it is therefore appropriate to explain a few of their leading features. The most evident characteristic they have is the piston or plunger by which the water is moved through them; an inspection of Figure 28 will show the difference between the piston, plunger and outside-packed water ends.

The first is used in small pumps almost exclusively, for pumping gritty water, and for pumping against a long suction lift. The The second is the usual type in water-works service. The third is used where the pump handles very gritty water or works against unusually heavy heads, because any leakage through the packing can be detected at once and remedied more easily than is the case with the other types.

The valves used in water-works pumps are generally disks of India rubber, vulcanized enough to give firmness yet be sufficiently elastic to permit bending at right angles and retain their

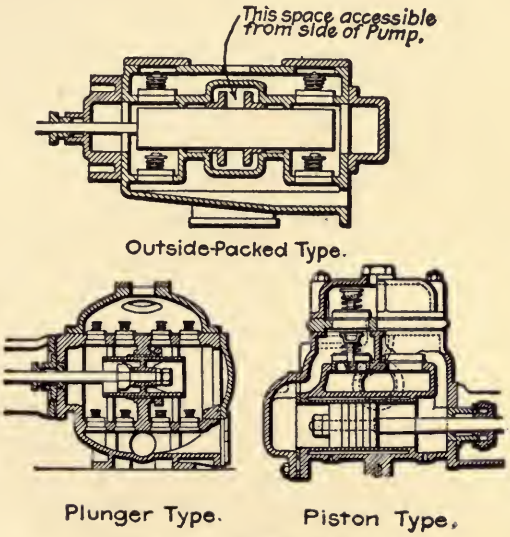


FIGURE 28.—TYPES OF PUMPS.

shape. They are held against their seats by coiled brass springs and move vertically on a spindle, which forms part of the seats. They vary in thickness from half an inch for 3-inch valves to three-fourths of an inch for 5-inch disks. They are a very important part of the pumps, for on their proper size and working depends in a large measure the satisfactory operation of the whole apparatus. In Mr. Barr's book on pumping machinery, previously referred to, the following statements are made: "The area of clear waterway through a set of valves in a water end should be not less than 40 per cent. of the plunger area for pumps having a speed of 100 feet per minute; and if that speed be increased to

say 125 feet per minute, then the combined water areas through the valve seats should be 50 per cent. of the plunger area; and in like manner 150 feet per minute would require 60 per cent. valve area, 175 feet per minute would require 75 per cent. valve area, and 200 feet per minute should have a valve area equal to the plunger area."

The pump valve problem is like the steam safety valve problem in certain respects, and there are many different opinions concerning its solution. The area of a circular port varies as the square of its diameter, so that doubling the diameter increases the cross-section of the waterway four times. Hence it would be an easy matter to furnish ample valve area were this the only factor to be considered. But after the water has passed through the passage it must flow out from under the disk, and the circumference of the circle around which it can escape increases only in the same ratio as the diameter. Hence when the diameter of a passage is made large it is necessary to give the valve covering that passage a high lift, and that is what many pump makers consider particularly undesirable. The quicker the action of the valves the smoother will be the operation of the pump, and it is for this reason that a number of small valves with comparatively low lifts are generally preferred to a few large valves with high lifts. Sometimes the total valve area, irrespective of the lift of the valves, is too small. Mr. Barr describes what happens in this case in the following words: "In a quick-running pump with too small a valve area an excessive lift is required of the valves, so that, in the interval of seating, a portion of the water in the pump cylinder passes under the valves and back again into the suction chamber; at the moment when the pressure overtakes the valves in their downward movement the velocity is so greatly accelerated as to force them violently down upon their seats, the pump becomes noisy, and nothing will relieve the pump but a reduction in the speed of the plunger, suited to the proper and noiseless action of the valves. Noisy action is not always confined to quick-running pumps; it is a common fault with nearly all low-priced pumps, the temptation evidently being to put in larger water plungers than the valve area can supply at the common rating of 100 feet piston speed per minute."

The unique form of valve designed by Prof. Riedler is free from many of the limitations of the usual disk valve, and merits

special mention. It is a large circular valve with a lift of 1 to 2 inches and replaces the entire deck of valves in a water end of the customary type. It opens automatically at the beginning of a stroke, and remains open until the end, when it is closed positively by an automatic device. In this way no limitation is placed on the piston speed of the steam engine by slow valve action, and the whole pumping apparatus can be run at the speed which gives the greatest economy in the consumption of steam.

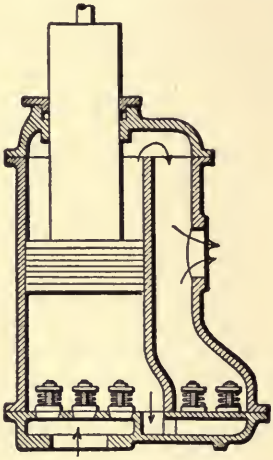
The water end should be provided with valves for draining it, and a connection for a charging pipe, through which water is admitted from the force main into the suction chamber, in case the charging pipe is not carried directly into the suction pipe. In crank pumps a by-pass connecting the two ends of the water cylinder is often employed, particularly on direct-pumping service. This by-pass is merely a small pipe from 1½ to 3 inches in diameter, by which part of the water forced ahead by a plunger can return into the cylinder behind the plunger, thus relieving the latter of considerable work yet allowing a higher piston speed than would be possible otherwise. In case a compound pump is working against a high pressure, such a device enables the engineer to start under a comparatively light load and run in this way until the steam end is in a condition to take the maximum load. With fly-wheel pumps working under very much lighter loads than they were designed for, the opening of the by-pass prevents the irregular operation which is a defect of some pumps of this class under such conditions. The charging pipe from the delivery main, previously referred to, sometimes connects with the pipe forming this by-pass, and the combination is often called the priming pipe in consequence.

SPECIAL POWER PUMPS.

Power pumps of small capacity are now generally made in a very different form from that of the water end of a direct-acting pump. Three cylinders, usually vertical, are employed, and for this reason the pattern is called triplex. This type of pump has been tried by many years of service under harder conditions than those of town water-works and has grown steadily in favor. The greatest trouble with them in the past has probably been lack of sufficient valve area, although complaint has been made against the failure of some makers to provide means of taking up wear in some parts of the apparatus. This has prevented running at

high speeds without pounding, and hence restricted the capacity of the pumps to lower amounts than German pumps of the same stroke and diameter of plunger. However, the attention paid to such machinery in the last few years has resulted in marked improvements.

Until within a short time the three cylinders of these pumps were single acting, and each discharged water during but half a revolution of the shaft driving its plunger. The three cranks on the shaft, which drive the plungers, are placed at angles of 120 degrees with each other, so the combined discharge of the three cylinders, even when they have single acting plungers, is



Differential Pump.

FIGURE 29.—DIAGRAM OF CONSTRUCTION.

approximately uniform. Triplex pumps have lately been put on the market in which this discharge is still more uniform; this is obtained by the use of differential plungers. As these are also used extensively in some of the latest sypes of vertical engines, a diagram of their action is shown in Figure 29. The upward stroke of the piston draws the cylinder full of water. The downward stroke forces this water through the passage at one side to the discharge opening, where some of it, usually one-half, is driven into the force main and the remainder passes into the top of the cylinder above the piston. The piston is surmounted by a large plunger which occupies a considerable portion of the cyl-

inder space, and hence does not let all the water discharged by the downward stroke of the piston into this space. On the next upward stroke of the piston this water above it is driven out while a fresh supply is coming into the lower part of the cylinder through the suction valves. A water end of this type must be made very strong because of the abrupt change in direction of the flow through portions of it, which change exposes all the moving parts to shocks of considerable magnitude when the pump is working against high pressures.

A form of power pump which has made some strong friends among mining and irrigating engineers but has only recently been entered as a competitor among water-works apparatus is the rotary. The construction of these pumps is so well explained in the trade publications of the makers (see the advertising columns of "The Engineering Record") that it is unnecessary to refer to the matter here. A pair of these pumps was installed for the Connersville, Ind., water-works in 1888. One of them is used for domestic pressure, 45 to 60 pounds, and the other is run when a fire pressure of 100 to 120 pounds is needed. Each is driven by an independent turbine water wheel, that for the fire pump being considerably larger than the other. After running about five years the fire pump was overhauled at the request of the water committee, as they wished to have the apparatus in the best condition while testing some new water mains. No repairs were needed, however, until the plant had run about nine years, when the shafts had to be replaced. The river water supplied to the city is at times quite gritty, but in spite of this the revolving pistons had worn but 0.03 inch in their nine years of service. The plant requires very little care. For several years, possibly at present also, the manager of the plant worked full time in a furniture factory adjoining the pumping station, and the pumps looked after themselves. He oiled them every morning before going to work, and in case a fire alarm was sounded his wife, who lived in the building, went down stairs and started the fire pump.

Another form of power pump, known as the screw pump, is giving good satisfaction under pressure of less than 150 pounds. It is comparatively new and untried for water-works purposes. Its maker is selling it under very rigid guarantees as to its efficiency, and the larger sizes seem suited for use in the class of small works described in these articles.

The centrifugal pump is rarely used in water-works for the main pumping engines, owing to its limited forcing power, but it is admirably adapted for raising large volumes of water short distances, such as from rivers or ponds to sedimentation basins, filter beds, or the pump wells of main engines where very long or otherwise unsatisfactory suction mains are necessary without such preliminary pumping. When well designed and not overloaded their efficiency compares favorably with that of other power pumps working under similar conditions, and the fact that grit and sediment in the water have little effect upon them renders them especially useful in handling such water before it is clarified. Where the pump is more than a few feet above the water it is best to use what is called the double-suction type, as a greater efficiency is probably obtained with it than with the more common type. Centrifugal pumps are proving useful at the bottom of open wells, which, for one reason or another, have been reinforced by driven wells sunk in their bottoms. These wells are connected to a centrifugal pump at the bottom of the open well, and in this way the volume of water is largely increased at a comparatively small expense.

Deep-well pumps are often operated by working heads, so called, driven by stationary engines, and where economy in the use of fuel is particularly desired their use is advisable. The ordinary direct-acting deep-well pump is very uneconomical in the use of steam, as already mentioned, compared with a slide-valve engine of the same horse-power, and for this reason the additional cost of the plant, consisting of slide-valve engine, belted or connected by a friction clutch to a working head, may be more than met by the reduced fuel expenses. The working head is also useful where a gasoline engine furnishes the power, a rapidly increasing practice.

The principle of the working head is very simple, merely the conversion of the revolving motion of a pulley into the reciprocating motion of a plunger working in a drop tube run down the well, or sometimes in the tubular casing of the well. There are a large number of such devices on the market, differing only in small details. In selecting one care should be taken that the wearing surfaces are large and well oiled and all the parts are strong and heavy, weight being of importance on account of the tendency of the head to shake if it is light and working over a

well of considerable depth. For many years these heads had a single plunger which delivered water on the upward stroke only. Afterward the top of the plunger rod was provided with a smaller plunger, the two making a differential plunger. This arrangement furnished a fairly continuous flow, but no more water was obtained than with the original device. To remedy this two plungers were introduced. The lower one has a solid rod running down from the head, while the upper one is provided with a hollow rod in which the other moves. The working head in this case is arranged so that one or the other piston is rising practically all the time and the discharge from the well is greatly increased. A recent form of these heads, called a continuous-flow head, is designed to give a faster speed to the downward than the upward stroke of each plunger, and thus render the flow more uniform than with the older type. The construction of these various forms of heads and the plungers they operate are so clearly shown in the trade publications of the makers that it is unnecessary to illustrate them here.

The air-lift pump will be referred to in the next chapter.

It sometimes happens that a small district which must be supplied with water is much higher than the remaining portion of the town. To supply all the consumers with water under the pressure necessary to reach these few elevated houses may be undesirable on account of the heavy pressure it will put on most of the fixtures, and, if all the water is pumped, on account of the disproportionate expense. In such a case several plans may be followed. If the town has an electric lighting plant its simplest plan will be to install a power pump driven by an electric motor controlled automatically in much the same way that some elevator pumps are now controlled. This pump would take its supply from the nearest main to the hill. Of course, a steam or gasoline engine might also be used, and a tank could be built to store enough water at or near the highest point to prevent any annoyance in case of accident to the pumping machinery. In case the hill is on the line of the force main or the chief main in the case of a gravity supply, a water motor might be used to pump the small supply needed in the high service district. There are several types of these motors on the market, some of them, like that used in New London, Conn., described in "The Engineering Record" of October 11, 1890, being very interesting mechanically.

SETTING PUMPS.

The setting of the pumps and their connection with the suction and delivery pipes is generally done by the manufacturers, where the apparatus is of fair size, say 500,000 gallons or more in 24 hours. They will furnish drawings showing the size of the foundations needed, and these are often built by the masons who put in the foundations of the pumping station, but if the pumps are erected by the manufacturers it is advisable to put the responsibility of the foundations on them also. There can then be no question as to who is liable for any failure in the operation of the machinery.

The manufacturers also furnish tables showing the smallest diameter of suction main to be used with any pump, and if this main is to be more than 50 feet long or to have any bends in it, they should be asked to select the proper diameter, unless the main is part of a driven-well plant, in which case it should be designed so that the greatest velocity of the water in it at any point does not exceed 100 to 150 feet per minute, depending on the readiness with which the wells yield the water. This limit to velocity is only one-half to two-thirds the usual velocity in suction pipes, but it is desirable with tube wells in order to keep the friction losses between the well strainers and the pumps as small as possible. Flanged cast iron pipes make the best suction mains. They should be laid with the greatest care to prevent future sinking at any point, and there must be no vertical bends in which air can collect. They should have a uniform upward grade toward the pumps of not less than 6 inches in 100 feet, and, after laying, be tested for tightness under a pressure of 40 or 50 pounds. A leak, no matter how small, affects the pumps badly. It is usual to put a foot-valve at the inlet of the suction pipe when this ends in a well, in order to keep water in it while the pump is idle, and, in case the pumps work against heavy pressure, the suction pipe should also have a relief valve. This prevents serious pressure coming on the suction pipe in case the pumps are stopped, and leaky valves allow the pressure in the force main to be communicated through the pump to the suction main. A strainer is used in the suction main whenever there is any probability of leaves or similar objects reaching the pump; this strainer is placed either at the end of the suction pipe or near the pump, the latter location making it easier to clean out any obstacles which may be caught.

If the suction pipe is long or the difference in elevation between its ends is considerable, it should have a vacuum chamber as near the pump as possible to keep the flow in the suction main uniform, and prevent hammering in the water end. The pump is similarly provided with an air chamber to steady the flow in the delivery main. These are made of copper for small pumps and cast iron for the larger sizes, and are too often little more than a hollow mockery owing to the failure of the manufacturers to provide gauges and charging apparatus to keep them full of air. The capacity varies "from four to fourteen times the capacity of the barrel of the pump," increasing with the pressure against which the pump works; there are a few manufacturers who do not favor them except under high pressures, but the majority regard them as valuable under all conditions. The vacuum chamber, when one is used, is generally about half the size of the air chamber. An important paper on air chambers will be found in "The Engineering Record" of September 13, 1890.

The force main should have a gate valve and a check valve, the latter as near the pump as possible, to keep the pressure in the main from the pump while it is idle. A small charging pipe, already mentioned, is attached to the force main beyond the check valve and ends either in the suction main or the suction chamber of the pump; its purpose is to charge the suction pipe with water when the pump is to be started. In order to allow all air which may have found its way into the pump while it was idle to escape and so prevent the jamming of the valves, it is necessary to have the check valve in good condition and to provide a small starting or waste delivery pipe through which the mingled air and water of the first few strokes may be discharged. It is shut off as soon as the pump begins to run smoothly. These different attachments should be furnished by the manufacturers of the pumps and set up by them. The details of the steam piping and frequently of the boiler plant, are left to them, although in such cases it is customary in advertising for bids, to state that any or all bids may be rejected, in order to enable the water commissioners to reject low tenders based on unsatisfactory apparatus. It is a better plan, however, to retain an engineer having some experience with pumping plants to outline the features of the desired plant in a general way in the specifications, and determine on the test necessary for its acceptance. This enables the bidders to

figure on about the same class of machinery and workmanship, while allowing them sufficient latitude to use their special features of design. As regards tests, they should be as simple as possible for the class of works under discussion. In case the pumping machinery is purchased under a guaranty, a test should of course be made to determine whether that guaranty has been carried out. It is ridiculous to specify certain requirements for pumps and then fail to ascertain whether such specifications have been fulfilled.

CHAPTER XIII.—THE AIR LIFT.

The air-lift method of raising water is by no means new, although made a commercial success within a comparatively few years. It is based primarily on the fact that, when air is passed into the bottom of a tube submerged in water, the water level in the tube is raised. In practice the wells are piped in various ways. Sometimes the air pipe passes down the center of the well, and the water is forced up between its outside surface and the casing; sometimes the air pipe is hooked at the bottom, and its upward-pointing nozzle is inserted below the open end of a discharge pipe; sometimes the discharge pipe is inserted within the air pipe, and the air passes down through the annular space between the two to suitable orifices near the bottom of the inner tube; sometimes the air and discharge pipes are connected by a semicircular bend, from the lowest point of which a single pipe is dropped farther down the well. All these methods are further modified by the use of special nozzles and patented features controlled by the various contractors. The latest of these patented piping arrangements with which the writer is acquainted, consists of the well casing and another tube nearly as large and extending into the water a considerable distance, the space between the two forming a passage for air. Inside the second tube is a much smaller one, down which still more air can be forced, the water being driven up between it and the intermediate pipe. The small central pipe has apertures at different elevations which can be opened separately in groups, or all at once, by an ingenious arrangement of valves operated by a single stem and a handwheel at the top of the well.

The arrangement of the details of the piping is always left to the contractors, for their practical knowledge generally enables them to accomplish the required result with the minimum amount of experimenting. The theory of the air lift is chaotic, and theoretical designs have to receive considerable practical correc-

tion before they will work satisfactorily. Contractors do not care to call general attention to their achievements, as their competitors will then visit the plants and learn the secrets of the success. There are some remarkable plants in this country, of which it was hoped the builders would agree to furnish reliable information, but they have all declined for business reasons.

Prof. Elmo G. Harris, of the School of Mines at Rolla, Mo., pointed out clearly the difficulties in the way of a satisfactory theoretical discussion of the air lift, in the "Journal" of the Franklin Institute for July, 1895. The air pressure was shown to depend on the depth below the quiet water surface at which the air was discharged, and on the velocity of the water at this point. The slip of the air lift is a very important factor; it is the difference in velocity of the air bubbles and the water in the discharge tube, and depends in a large measure on the volume of each individual bubble and hence on the form and area of the air inlet. The net lift is, of course, an important factor, and the area of the discharge pipe also affects the results. Finally, the ratio of the expansion of the air as it rises through the main pipe and the total volume of air in this pipe cannot be changed without affecting the efficiency of the plant. In a general way it may be said that the present practice is to submerge the main pipe about a third or half again as deep as the net lift from the water to the point of discharge, and to use just enough air pressure to overcome the pressure of the water column. More pressure will give more water, but the cost per thousand gallons will be increased. The piping is generally altered a number of times before the best form is determined. In case a number of wells are to be pumped, it is strongly recommended that a valve be placed on the air line near each well rather than near the receiver from which the air is delivered.

The air compressor for an air-lift plant should be selected carefully, as upon it will depend in a large measure the satisfactory operation of the station. It would be out of place to discuss compressors in this place, particularly as Mr. Frank Richards has already done this in his valuable book entitled "Compressed Air," but a few general statements are necessary for an elementary discussion of the subject in hand. In the first place, when air is compressed, part of the work done in the steam cylinders is lost in a number of ways. From 5 to 20 per cent. may be spent to

overcome the friction of the compressor, and it is here that good workmanship keeps down the waste. From 5 to 20 per cent. more may be lost in heating the air in entering and by clearance, a loss which good design diminishes. Finally from 15 to 35 per cent. is inevitably lost by heating the air during its compression, for it is impracticable to keep the air at a constant temperature while its volume is being reduced and its pressure raised. The last loss is an important one when the compressed air is to be used to lift water.

Just how much drop there is in the temperature of the compressed air from the time it leaves the cylinders until it mingles with the water in the well depends on a number of circumstances. Where the air is compressed to 75 pounds and the water in the well is at 45 degrees Fahrenheit, the fall in temperature is about 350 degrees. It has already been stated that part of the work done in the steam cylinders is spent in raising the temperature of the air, so if the latter is subsequently cooled 350 degrees, all the work done in raising its temperature this amount is necessarily lost. Under the conditions mentioned the loss will be about 23 per cent. For many purposes to which compressed air is put this waste of energy is partly avoided by reheating the air before using it, but this remedy is useless in air-lift plants.

It is evident that in stations for pumping water it is particularly important to keep the final temperature of the compressed air as low as possible. There are three means of doing this.

The first is by cooling the air during compression, by selecting suitable forms of compressors. Such machinery may be classed as wet or dry, according as water is or is not present in the air cylinders during compression. There are two distinct types of the wet compressor. In the first the water enters the cylinder with the air, and in the second it is sprayed through the air during compression. The first type is inefficient because the water does not cool the air very much and merely runs in and out of the cylinder as if the machine were a pump. The second type is the most efficient compressor there is, so far as delivering cool air is concerned, but the parts wear out too rapidly to make it a commercial success. American compressors are now of the water-jacketed type, and what cooling is accomplished is effected by circulating cold water over the outside of the cylinders in which the air is compressed. They consequently give cooler air when

run at low than high speeds. The volume of a cylinder is proportional to the square of its diameter, while the cooling surface is proportional to the first power of the diameter, hence compressors built with a large number of small air cylinders immersed in water should give unusually cool air. Such machines have been built and give the expected results, but their complexity has prevented their general use.

The second method of keeping down the final temperature of the compressed air is by raising the pressure in stages and cooling it between the successive stages. It may be said in a general way that the best current practice is to use two stages when the final pressure is between 60 and 300 pounds, and three stages when it is between 300 and 1,000 pounds. Two-stage compression to 75 pounds means a saving of about 17 per cent. over single-stage, and three-stage compression to 500 pounds means a saving of 25 per cent. over single-stage.

The third method is to take the air as cool as it is possible to obtain it. About 1 per cent. is saved for each 5 degrees fall in temperature of the free air entering the compressors. During the winter months a considerable economy can be effected by drawing the air from outside the station building, but if this is done care must betaken to make the flues to the inlets large and free from bends. The writer knows of no air-lift plant where this is done, and it would probably prove an uneconomical refinement in many small plants. At La Grange, Ill., a connection was made from the well casing to a cylindrical drum about 18 inches in diameter and 8 or 10 feet high. From the side of this drum the water discharged from the well was taken, and from the top was led an air pipe to the intake of the air compressor. The air, after doing its work in the well, is reduced to the temperature of the water, and is returned to the compressor at about 30 degrees lower temperature than the engine room. A two-day test of the plant, the first day drawing the air from the engine room and the second day from the well, showed that about 6 per cent. less coal was needed when the air from the well was used.

A receiver should always be placed near the compressor to hold the air. It equalizes the work of the compressor in much the same way as the air chamber on a force main, and also acts as a separator to catch the water and oil which are carried by the air. These are blown out at frequent intervals. As regards the pip-

ing, the following hints are quoted from a lecture by Mr. William Prellwitz, of the Ingersoll-Sergeant Drill Company, before the students of Lafayette College: "All pockets in pipe lines should be avoided, as they have a tendency to hold water, and thus retard the free passage of the air. Should it become necessary to pass air through a pipe line which must of necessity have many pockts in it, these may hold so much water that very little air pressure will be had at the end of the line. Where these conditions exist much trouble can be avoided by thoroughly cooling the air, thereby taking out all its moisture, to a temperature lower than that of the pipe through which it will pass, so that the air will have a tendency to take up moisture in the pipe instead of dropping its water in it."

The efficiency of the air lift depends on so many factors, which are only now beginning to be understood, that the apparatus has been regarded as extremely uneconomical. It certainly is not so economical for high lifts as some of the latest forms of power-pumps or as plunger pumps inserted in a pit. On the other hand, a deep well of small diameter costs much less than a larger one, and, if the well discharges freely, the extra cost of air pumping may not be so great as the extra cost of the large well. This is particularly important where the plant is a small one and is operated intermittently. A particularly useful field of the air lift is in gauging wells to ascertain their maximum yield, on account of the readiness with which the conditions may be changed by altering the piping or changing the speed of the compressor.

While the efficiency of the air lift is under consideration, attention is drawn to the accompanying table of the results of a number of tests conducted under the direction of Mr. F. A. W Davis, of the Indianapolis Water Company. The well was 10 inches in diameter and 330 feet deep, and was tested by measuring the time it took to fill a tank holding 267 cubic feet, under the conditions given in the table. The air was measured by a Wilie proportional gas meter and the figures give the volume of compressed and not free air. The temperature of the air as it came from the compressor was 176 to 177 degrees, and that of the water as it emerged from the well was 54 degrees. The aeration of the water by the lift caused it to precipitate the iron in solution, an advantage of this method of raising water noticed in other plants.

Test of Air Lift, Indianapolis.

Pumpage.			Air.		Piping.	
Time.		Discharge per minute.	Amount.	Pressure.	Depth in water.	Height above water.
Min.	Sec.	Cu. ft.	Cu. ft.	Lbs.	Ft.	Ft.
2	19.5	115	152	36	64.3	25.7
2	12	121	150	36	"	"
2	16	118	146	36	"	"
2	21	114	140	37	"	"
2	46	96	255	46	41.3	25.7
3	55	68	240	46	"	"
1	53	142	100	44	82.7	27.3
1	50	146	101	44	102.2	27.8
1	35	169	100	50	"	"
1	45	152	100	43	"	"
1	43	155	100	43	"	"
1	45	153	100	45	"	"
1	37	165	100	44	"	"
*1	55	139	106	45	83.6	27.3
*1	54	141	107	45	"	"
**1	48	148	99	43	80.7	27.3
**1	47	150	96	46	"	"

Note.—The first 13 tests were made with a 2½-inch air pipe; those marked * with a 4-inch pipe; those marked ** with a 2½-inch pipe having a curved nozzle deflecting the air upward.

At a meeting of the Western Society of Engineers, held March 3, 1897, there were many statements made concerning the field of the air lift and its advantages, to which attention is drawn. The full discussion will be found in the society's "Journal" of April, 1897.

Mr. Thomas T. Johnston.—"In almost all cases the air lift is used to raise water to the surface of the ground and no higher. It might, in some cases, be used to raise water to a tower above the ground, but its efficiency falls off so rapidly as the head pumped against increases, that it is better to raise the water to the surface only, and to pump from the surface to the stand-pipe or into the mains by ordinary plunger pumps. It is also necessary to allow the water to free itself from air before it enters the mains. Since the practical case may be assumed to involve lifting water to a height above the surface, an air-lift plant will be considered to include not only the compressor, receiver, piping, reservoir, etc., but also the pump and appurtenances. The efficiency of the plant must involve the expenses due to double pumping.

"It has been a common claim for the air lift that wherever applied it has resulted in an increased flow from the well. This has undoubtedly been the fact in a great many cases. Take the case

of the well in question, for instance, and the diameter of the well tube to be anything less than 12 inches. It would not be possible to insert in the tube either an old-fashion or continuous-flow deep-well pump having a capacity of 500,000 gallons per day. Neither of these pumps could reduce the pressure at a point 50 feet below the surface to that of the atmosphere. The air lift could do so, however, and would thus be able to derive more water from the well than either of the other pumps. If, however, the well tube were 15 inches or more in diameter for 30 or 40 feet below the surface, then the continuous-flow pump would come into the field and raise the water with much higher economy than the air lift. Or, if the conditions be favorable, plunger pumps in a pit 30 feet deep would do the same thing with more economy doubtless than either the air-lift or continuous-flow pump. There is nothing inherent in the air lift that causes an increased flow from a well, though under certain conditions it is capable of producing that result. The continuous-flow pump method requires only a sufficiently large well tube, and the pit method requires only a sufficiently deep pit.

"It must be evident that no general rule can be laid down to determine the best method of pumping to adopt for any or all wells. The conditions surrounding any particular case may vary so widely from those of another case that what may be suited to one will not be of any use for the other. It can be said in general that when large water supplies are involved, for cities of 20,000 inhabitants or more, and where atmospheric pressure in wells must be made to occur at levels below the surface of the ground, the pit method is the most advantageous. For very small supplies, where the total amount of money involved is not a large item in any event, and where economy in operation is not a matter of moment, the air-lift and old-fashion deep-well pump may find useful application. The intermediate field affords opportunity for strife between the pit method and the continuous-flow pump in a degree varying with the several conditions of any particular case."

Mr. J. F. Lewis.—"The advantages of the air lift are that the machinery is all above ground and concentrated so as to require less attendance than any other system, a great increase of water, and no repairs whatever. If the water falls in the well it can be followed down, and, by installing an economical compressor with

a compound condensing Corliss engine and compound air cylinders, with the new systems of piping the wells, there is no question but what the cost per million gallons will compare favorably with any system that is in vogue."

Mr. D. W. Mead.—"The air-lift system has its chief use when the quantity of water is the chief concern, and where this is to be taken from single wells and the cost of operation is not a large consideration. It is the best combination pumping appliance which has been placed on the market for obtaining a large quantity of water from a small hole. For instance, some preliminary experiments were made with the air lift on a well at Rockford, Ill. This is an 8-inch well drilled into the Potsdam sandstone and flows at the surface. It flowed at the outlet of the discharge pipe about 150 gallons per minute. When the air lift was started with a 6-inch pipe inside the 8-inch casing, the amount of water was increased to about 650 gallons per minute, and when the 6-inch pipe was taken out and the 8-inch casing used as the discharge outlet, the discharge was increased to over 900 gallons a minute. It is probably impossible, with an ordinary power pump, to obtain such a large dicharge from a small hole as this experiment shows, and, in places where it is a question of volume and not economy, the air lift has this advantage over anything else that has been offered."

CHAPTER XIV.—PUMPING STATIONS.

In designing a pumping station it is not enough to provide machinery which will furnish a certain quantity of water daily. That is easily done, but it is not such a simple task to design works which will be the most economical in operation during the whole of their period of service. Such a plant is laid out with a view to using the most available fuel in the most economical manner, and the utilization of the steam by engines and accessories of such types that any further additions to the plant will cost more than they will save. The evaporative efficiency of various coals ranges widely and the furnaces to get the best results from one fuel are different from those for another; hence the best plant for one locality may be far from the best for another, although the amount and pressure of the steam are the same in both cases. Before the plans of a permanent power plant are finally adopted, it is therefore very desirable to have them examined by some one thoroughly acquainted with such work, for his suggestions may result in a saving of far more money than his fee.

While it would be useless to attempt in this place to discuss the economical features of the different classes of pumping machinery, nevertheless the stations themselves may be considered in a general way. In the first place, it is important to determine the advisability of combining a water-pumping and electric station in one plant. This is now frequently done, as at the Greenwich, Ohio, plant shown in Figure 30, which was designed by Mr. Burton J. Ashley. The water at this plant is forced by a 500,000 gallon pump to a 100 x 11-foot standpipe. There is a 100-horse-power engine running at 180 revolutions per minute and belted to a 40-light dynamo for arc lights and a 500-light dynamo for the incandescent service.

Between such a plant and the pumping station at Walpole,

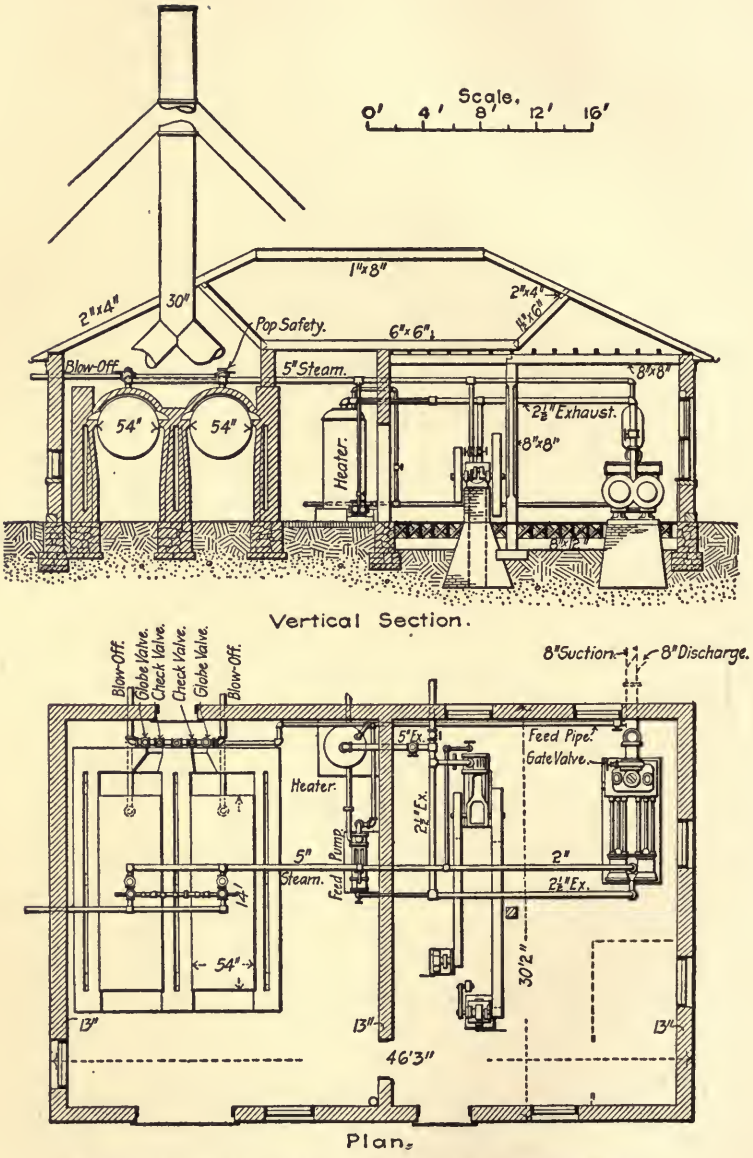


FIGURE 30.—WATER AND ELECTRIC STATION, GREENWICH.

Mass., shown in Figure 31, there is a wide variety of intermediate types which it would be fruitless to discuss. The Walpole plant, designed by Mr. Freeman C. Coffin, is a building with 14-inch brick walls and a slate roof carried by wood trusses. The floor of the engine room is 6 inches of hydraulic cement covered with 1 inch of asphalt. The boiler room has a floor of hard brick laid on edge in cement on 3 inches of sand. The water is drawn from driven wells through a sand catcher into a surface condenser and thence to the pumping engine. The sand catcher also serves to intercept the air from the wells, which is removed by the small air pumps mounted on its top. The steam from the feed, air and main pumps passes through a horizontal heater, and then through the surface condenser before it is pumped into the hot well. In this plant a duplex pump is used; if a fly-wheel pump were employed a separator would be placed on the steam pipe to prevent water entering the cylinders, which is a serious matter with such pumps, but less important with the duplex type. The carpentry and masonry in such a building are matters with which every builder is acquainted and do not require discussion.

Where vertical pumps are employed a dry well is necessary to contain the water cylinders, and this sometimes involves more trouble in construction than any other portion of the plant. If the amount of water which it is expected to encounter is large, it may be advisable to sink the wall on a curb in the manner already described for open wells, laying the masonry with great care as the curb sinks. When the necessary depth has been reached the bottom can be dredged out to a fairly level surface and then covered with quick-setting Portland cement concrete deposited under water by buckets or otherwise. If the work is done carefully and no attempt made to pump out the water until the concrete has set, it will very likely be possible to make a fairly dry well in this way which can be made water-tight afterward by a rendering of cement, an asphalt coat or a carefully laid brick lining. A number of hints for the use of asphalt as a water-proofiing material are given in Chapter XVIII. under the head of asphalt-lined reservoirs.

What is known as the Sylvester process of water-proofing is sometimes used for such work. The dry-well of the new Steubenville water-works, designed by Messrs. Wilkins & Davison, affords an instance. The wall of this well is 3 feet thick at the bottom

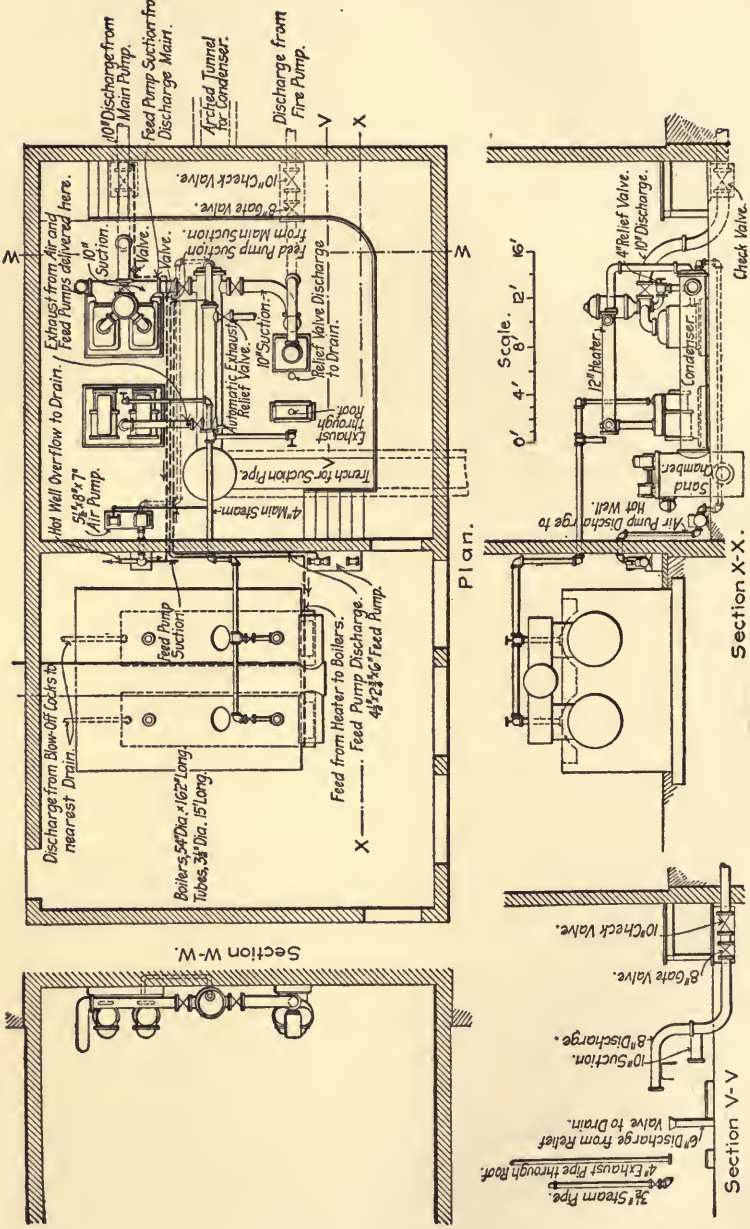


FIGURE 31.—PUMPING STATION AT WALPOLE, MASS.

and 2 feet at the top, and consists of a 9-inch outer ring, a 2-inch filling of Portland cement mortar, and an inner ring. The specification for the water-proofing reads as follows: "When the walls have thoroughly dried out the inside shall be coated with a solution of Castile soap, three-quarters of a pound to one gallon of hot water. This shall be applied while hot. After remaining 24 hours there shall be applied a second coat composed of half a pound of alum to 4 gallons of water, then another coat of soap solution, and finally a second coat of alum solution, 24 hours to elapse between each coat."

So little has recently been printed about this process that it may be well to reprint in this place a description of its use in water-proofing a gate-chamber of the Croton Reservoir in Central Park, New York. This was written by the late W. L. Dearborn and printed in the first volume of the "Transactions" of the American Society of Civil Engineers. The gate-chamber in question was built of the best quality of hard-burned brick laid in a mortar of one part Rosendale cement and two parts of sand. Concrete filling was placed between the brick walls and the latter were laid with great care, but when the chamber was put in service under a head of 36 feet water passed through the masonry in considerable quantities. Mr. Dearborn's description of the process of water-proofing is as follows:

"The process consists in using two washes or solutions for covering the surface of the walls—one composed of Castile soap and water, and one of alum and water. The proportions are three-quarters of a pound of soap to one gallon of water and half a pound of alum to 4 gallons of water, both substances to be perfectly dissolved in water before being used. The walls should be perfectly clean and dry, and the temperature of the air not below 50 degrees Fahrenheit when the solutions are applied.

"The first or soap wash should be laid on when boiling hot with a flat brush, taking care not to form a froth on the brickwork. This wash should remain 24 hours so as to become dry and hard before the second or alum wash is applied, which should be done in the same manner as with the first. The temperature of this wash, when applied, may be 60 or 70 degrees, and it should also remain 24 hours before a second coat of the soap wash is put on. These coats are to be applied alternately until the walls are made impervious to water. The alum and soap thus combined form an

insoluble compound, filling the pores of the masonry and entirely preventing the water from entering the walls.

"As this experiment was made in the fall and winter, 1863, after the temporary roofs were put on the gate-house, artificial heat had to be resorted to to dry the walls and keep the air at the proper temperature. The cost was 10 cents per square foot. As soon as the last course had become hard, the water was let into the bays and the walls were found to be perfectly impervious to water, and they remain so in 1870, after about 6½ years."

It has already been shown that pumps drawing their supplies from driven wells should be placed as low as possible in order to reduce the suction head to the minimum. A low suction head is far more important with these wells than where the supply is drawn from an open wet well, because of the air which is present in the ground water. Horizontal pumps were the only type formerly used for small works and an open pit large enough to contain them was frequently an expensive undertaking. The introduction of power pumps now makes it possible to keep the engines on the main floor of the building and put the pumps as low as seems desirable.

The pumping at Tarboro, N. C., affords a good illustration of such a plant. The old works owned by the town had a 40-horse-power boiler and a 12 x 7 x 12-inch duplex pump, the latter in a pit 5 feet below the floor line. The works recently passed into the hands of a private company and their present condition is shown in Figure 32. Mr. J. W. Ledoux, chief engineer of the American Pipe Manufacturing Company, which built the plant, informed the writer in 1897 that the water was drawn from eleven tube wells along the bank of Hendrick's Creek, and could be supplemented by a separate connection with the latter, which has a drainage area of nearly 10 square miles. The power is furnished by two gasoline engines of 13½ actual horse-power each, running at 250 revolutions per minute and belted to two 7 x 7-inch triplex pumps running at 40 revolutions per minute. The pumps were placed on a level with the top of the driven wells and the engines are 6 feet higher, on the same floor line as the steam pump.

The pumping chamber has a semi-circular 22-inch brick wall laid in Portland cement, and the floor is 2½ feet of concrete finished with pure cement. Being close to the creek, which often rises to a depth of 15 feet, the outside of the pumping chamber

was protected by 10-inch pine piles driven close together to a depth of about 15 feet. The building is hard brick with a metal-shingled roof.

All the stations constructed by Mr. Ledoux are notable for the care taken to make them attractive to visitors. The writer has been able to secure no satisfactory view of the interior of the Tar-

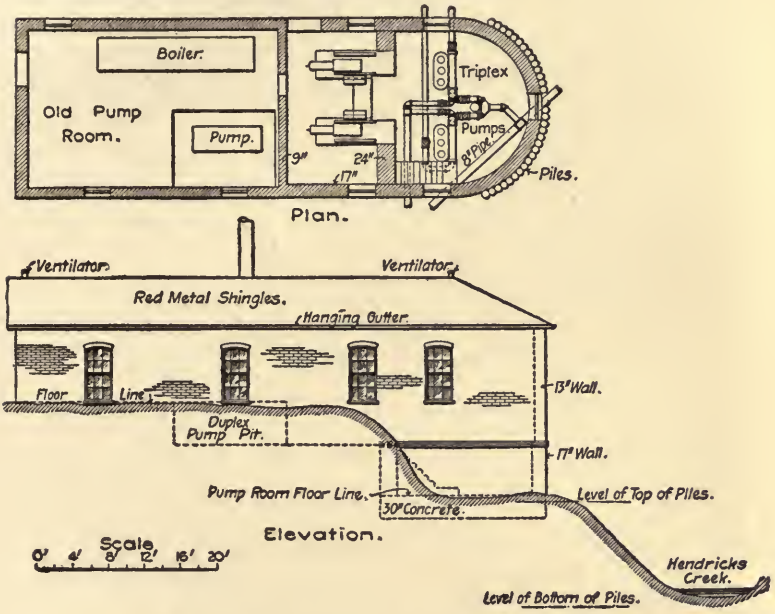


FIGURE 32.—TARBORO PUMPING STATION.

boro station, but Figure 33 shows the general finish of his plant at Westville, N. J. The contractor for the pumping plant at both these places, Mr. W. P. Dallett, used Otto engines and Deming pumps. The appearance of the engines suggests a show room rather than a money-making plant, yet it is highly probable such attractive surroundings lead the engineers to keep machinery in better condition than when it is placed in a frame building with a board floor and rough finish.

An innovation of recent date in lighting small pumping stations is the use of incandescent lamps furnished with current by a dynamo driven by a small impulse water-wheel. The water is taken

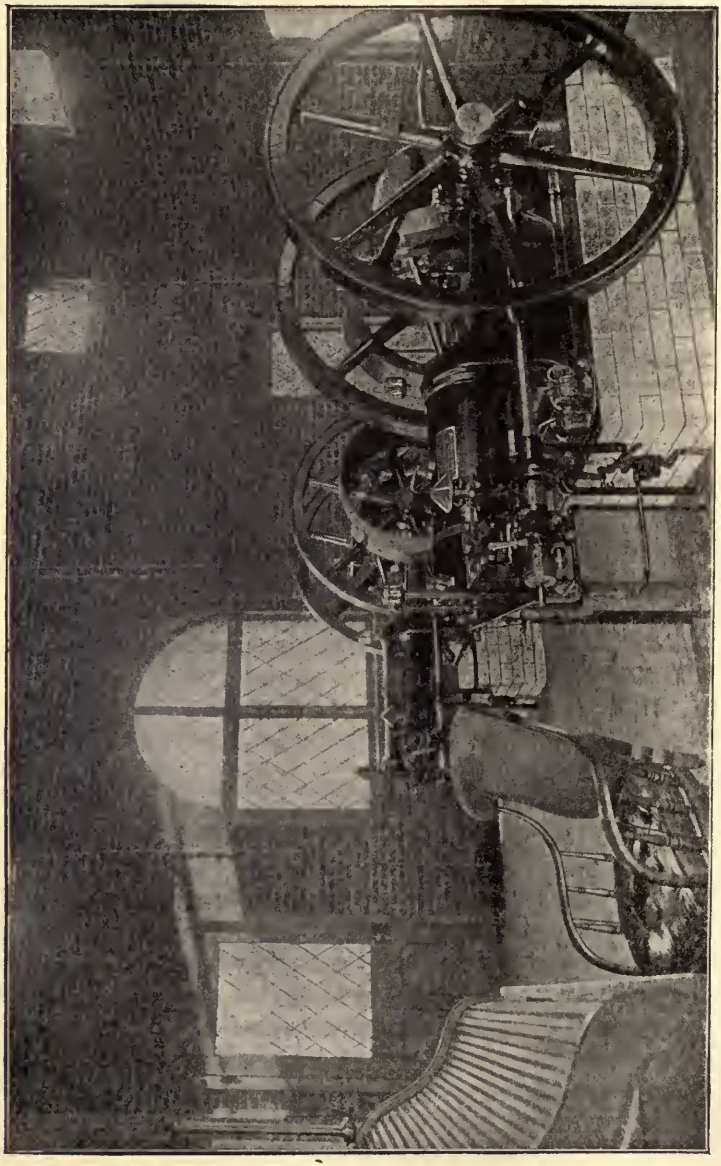


FIGURE 33.—INTERIOR OF PUMPING STATION AT WESTVILLE, N. J

from the main discharge pipe of the pumps and is wasted after use. Wheels of this sort are now built so that the size of the jet, and consequently the power developed, can be varied between wide limits. It seems to be pretty clearly demonstrated that where but a few lights are needed, such an equipment furnishes them at a very low cost.

CHAPTER XV.—INTAKES AND INTAKE PIPES.

Where water is drawn from a pond or river it is generally necessary to extend an intake pipe into the lake to an intake of some form. The intake may be connected directly to the pump and form its suction main, but it is better to construct a pump well or basin into which the intake pipe will discharge by gravity and from which an independent suction main may be run to the pump. The location of the pump well is determined mainly by three considerations. The first is the desirability of having the suction main as short as possible to keep down the friction of the water in passing through it. The second is the necessity of laying the suction main below the frost line in the earth and on a uniform rising grade to the pump. The third is the necessity of having the intake pipe discharge plenty of water by gravity into the well at all stages of the river or pond. The pipe offers a certain amount of resistance to the flow of water and allowance must be made for this. As a rule the amount of trenching necessary to bring the intake pipe into the well is much greater than that required to place the suction main below the frost level, and on this account it is desirable to place the well near the shore. The length of suction main to reach the well when so placed may, however, make it more advantageous to spend somewhat more money and locate the well nearer the pumping station. Under some conditions it may prove desirable to build a basin of some size rather than a small well, so as to have a supply of water sufficient for several hours or a day of maximum consumption. The conditions under which such a method of construction are desirable are so special, however, as not to warrant their discussion in this book.

The usual form of intake for small works is a strong timber crib filled with stone and having a vertical pipe rising a short distance

above its top. The bottom of the pipe ends in a quarter bend or tee by which it is connected with the end of the intake pipe. Sometimes the intake crib is not supplied with a vertical pipe, and the water passes down through a horizontal grating into a chamber containing the end of the intake pipe.

Where the current is sometimes strong and the bottom rocky, a trench should be blasted for the intake pipe. If it is not laid in a trench it is liable to be injured, and, if it is broken so as to be even partially stopped, the water-works may be seriously crippled for several days. The intake itself for such a situation is sometimes a well-braced sheet-iron box with perforated sides, securely anchored to the bed rock. The expense of having this work well done by a competent diver is comparatively large, but fully warranted if the local conditions render the usual intake unsafe. Where the river carries no silt, it may answer to blast a small pit in the rock bottom and let the intake pipe terminate in this, using a quarter bend and a short pipe extending downward and perforated on the sides if necessary.

The methods of laying intakes are legion. Some of the most interesting are described in the remainder of this chapter, and hints for other methods will be found in the section on submerged pipe of Chapter XVII.

Wooden stave pipe has been used for intake pipes on the Pacific Coast, and seems to be well suited for the purpose. The pipe is made by banding together carefully milled staves of redwood, Douglas fir or similar sound, clear wood. Intake pipes of such a type can be readily floated into place and anchored by timber cribs built over them and sunk by rocks. Care must be taken that no serious strain comes on the pipe at any place, and for this reason each anchorage crib should be hung by stout tackle from a scow or pile platform until it rests on the bottom. In one instance a wooden gate chamber was employed with such an intake pipe.

A form of intake used at several works drawing water from Lake Michigan was introduced, so far as the writer has been able to learn, by Dousman & Sheldon, of Milwaukee. It is a cast-iron cone with the intake opening at the apex. The intake at South Milwaukee has a diameter at the base of 15 feet and the exterior surface is curved to a radius of about 6 feet. The metal is 1¼ inches thick and is pierced near the bottom to allow the 12-inch

intake pipe to pass through the side and connect with a special casting inside the intake. This rises in a bell-shaped form at the top of the cone and is there covered by a cast-iron hemisphere perforated with ½-inch holes 1.2 inches apart. The intake is held in place on the bottom by riprap dumped over its flaring surface.

On account of the occasional stoppage of intakes by anchor ice, the plan has been followed in some cases of running pipes from the main intake to perforated cylinders or boxes 50 feet or so away. These are supposed to serve as supplementary intakes in case the grating over main entrance becomes clogged with ice. The plan seems to work successfully, which may be due to the fact that water is taken at points so far apart that there is no appreciable current toward any of them.

Filter-crib inlets in the bottom of rivers have been used by James H. Harlow, M. Am. Soc. C. E., at a number of places in the Pittsburg region, where the water contains so much silt at times that special precautions must be taken to keep it out of the pipes. A basin is dug at some point in the bottom where the current is strong enough to keep the larger suspended material from settling. A timber crib is then sunk in this basin and covered with 4 feet of stone, gravel and sand so as to form a filtering material which will intercept the silt. It seems improbable that such filtering cribs effect any marked improvement in the chemical or bacteriological character of the water, but their extensive use along the Allegheny River may be considered proof that their primary purpose, the prevention of turbidity, is attained. The crib differs materially from the older filter gallery in having a much greater surface and much less thickness of material through which the water percolates. Provision is usually made for washing them by reversing the current through the intake pipe; this is accomplished by putting a by-pass about the pumping plant and allowing water for the flushing to flow backward from the reservoir or standpipe.

The intake system of the water-works of the Ivorydale manufacturing establishment is one of the most notable instances of the filter system with which the writer is acquainted. About 1,000,000 gallons daily are drawn from a muddy stream which furnishes water unfit for use in its natural condition. Rows of 3-inch oak sheet piling were driven into the bed of the creek

so as to enclose an area of about 4,000 square feet. The mud was removed from this area, leaving a smooth hard clay surface, on which was laid a network of 6-inch drain tiles with open joints, which finally united at one point of the enclosure with ten lines of 6-inch tiles forming the effluent conduit. The enclosure was then covered with 2 feet of broken stone, 1 foot of gravel and 1 foot of sand. While such an intake does not appeal very strongly to the engineer in the light of present knowledge, it was apparently never intended to act otherwise than as a strainer, for the water flows through the effluent pipes into a pump well whence it is forced to an open sand filter for real purification.

The new intake of the Salem, Ore., water-works is another example of the submerged crib type. It is 20 x 60 feet in plan and divided by cross timbers into twelve compartments. It is sunk in a sand bar of the Willamette River, which is separated by a 2-foot layer of hard-pan from a stratum of coarse, water-bearing gravel. The top and upper part of the sides of the crib are sheathed with a double layer of 2-inch plank which renders it probable that most of the water drawn from this source comes from the gravel.

A special form of intake used at the end of a 24-inch intake pipe laid in 1894 at Burlington, Vt., is a copper cylinder attached to the end of an upright pipe held firmly by a pile of riprap stone around it. This cylinder is considerably greater in diameter than the vertical pipe and perforated on the top, side and overhanging bottom. The bottom is also fitted with hinged shutters folding upward, which are intended to allow water to pass into the intake pipe even when the perforations are closed by anchor ice.

The fear of anchor ice is ever present during the winter with superintendents of works in Northern States, and they have adopted a number of methods of protection against stoppage of the intakes. A frequent and serviceable one is the use of a small pipe run through or alongside the intake pipe to the screen at the intake. In case ice clogs the screen steam is turned into the pipe from the boilers and the ice blown away or melted. Compressed air has also been used for the same purpose. When it is employed, the entrance to the intake is a series of flaps opening upward, and so arranged that in case their perforations become clogged, the compressed air blown against their lower surfaces not only

lifts them but also blows the ice away from the holes. Mr. Joseph G. Falcon has carried the use of compressed air still farther in a revolving intake he has built at several places. The water enters the intake pipe through a revolving drum which is rotated about a horizontal axis by the compressed air before it escapes. The device is patented and details of its construction can doubtless be obtained from the inventor, whose address is Evanston, Ill.

The most remarkable combined suction and intake pipe with which the writer is acquainted is at Auburn, N. Y. It starts from deep water in Owasco Lake and runs up hill and down dale with entire disregard of gradients to a pumping station 9,560 feet distant from the intake. The pipe is 24 inches in diameter, and has numerous valleys and summits, one of the latter being 18 feet above low-water level in the lake and one of the former a submerged line under a stream near the pumping station. The reason for adopting such a remarkable profile was the desire to save expensive trenching. No suction pipe having such pockets for air can be operated without assistance, and in this case there is an air pump in the pumping station, connected with an air pipe line running back 7,800 feet over the top of the suction main. The two are connected at every summit and when the air pump runs there is no trouble with the suction main. If the air pump is shut down for more than a couple of hours or so trouble ensues. The writer believes such a plan deserves little consideration except in a few rare cases.

A form of intake which has been used at Newburgh, N. Y., Jackson, Miss., and other places is designed to allow the supply to be drawn from different elevations. The intake pipe is usually carried on a trestle of some sort to the place where the supply is taken. There it ends in a T branch. Each side end of the branch is connected by a sort of stuffing box joint with a half bend. These bends are rigidly connected by flange joints to the ends of another T branch from which the intake pipe proper projects. This combination makes a movable joint about which the intake pipe can be swung in a vertical plane by means of a rope passing from its free end over a pulley on a post rising from the trestle. An illustration of the Newburgh pipe was printed in "The Engineering Record" of September 2, 1893. This has swinging arms from the sides of a wooden intake pipe. Somewhat similar joints are made by several firms for sewage tanks, and could

doubtless be employed in situations where the movable arm is not exposed to serious strains.

The extension of the 20-inch intake pipe at Sheboygan, Mich., was an instance of submerged pipe-laying by means of a derrick scow, dredge and diver. The original intake pipe ended in a T with a piece of pipe calked in so as to extend four feet above the bottom of the lake. The water entered through the open bell end of this rising pipe. The outer end of the T was plugged with a piece of wood covered with canvas and held by iron clamps. The extension was 20-inch cast-iron bell and spigot pipe, calked together in sections of four pipes each, except the first and last sections, which were shorter. Between each section was a ball and socket joint of the Walker type; the corresponding halves of each joint were calked with lead to adjacent ends of the sections, making them average 50½ feet in length.

The dredge dug a 4-foot trench on a line with the original intake pipe, and, to get close to the T without disturbing it, dug a trench at right angles to the line of the pipe as close as possible to the T, which was marked by a buoy. A trench was also dug close to the T and diagonally to the pipe line so as to give the diver room to calk the first joint.

The first section of the extension was 21 feet in length, with half a Walker joint at one end and a spigot at the other for entering the T of the original intake. A scow was used having a flat deck large enough to hold three sections of pipe, derricks and diving apparatus. Two derricks were provided, cleated at the bottom and guyed to overhang the edge of the scow far enough to lower the pipes without chafing them. The short first section was lowered by means of a 12-foot oak timber 6x8 inches in section, chained to the top of the pipe and fitted with a chain at its center for attaching the falls of the derrick. This section was lowered by one derrick and the spigot inserted and calked into the T by a diver, who used a piece of flattened 1-inch lead pipe for a gasket.

Meanwhile the dredge had worked ahead and on the following day three sections were laid. These were lowered from the scow by the two derricks by means of two pieces of oak, 12 feet long and 6x8 inches in section, chained at each end to the center of each length of pipe and provided with a chain at the center of each timber for attaching the derrick tackle. By this means the weight of the section was evenly distributed over four points.

Before lowering each section a wooden plug was bolted to the outer end, so that nothing could enter the pipes until the plug was removed by the diver when ready to connect the next section. When in a correct position the diver attached a block and falls to the bolt holes in the two halves of the ball and socket joint and the pipe was entered by this means by the men from the deck of the scow, the diver guiding it home by means of a bar running through corresponding bolt holes in the two sections. This method was followed in laying all the sections.

The intake pipe ends in an L and upright piece reaching 6 feet above the bottom of the lake. The L was framed in with timber so as to give it a broad base to rest on in the bottom of the trench. After it was laid a T was placed on the end of the upright piece, and held in place by a chain running through it and fastened around the bell of the L. Over the two openings of the T were placed screens of 1-inch iron bars half an inch thick and 6 inches apart, crossing at right angles and riveted together. These were held in place by a half-inch iron rod with nuts at both ends properly tightened, running through the center bar of each screen and through the tee. After this was done stones were placed around the L and upright as far as the bottom of the T.

The 16-inch intake pipe for the water-works of Geneva, N. Y., a place of about 8,000 population, runs from the pump well about 700 feet to a point in Seneca Lake about 600 feet from the shore, where the water averages 25 feet in depth. At the shore line there is a heavy masonry pier built about the pipe, and there are several ball and socket joints along its length. Where it rests on soft bottom, flat plank supports were attached below the bells by copper wire. The intake is a box of three-eighths-inch wrought iron plates; it is about 6 feet square and $3\frac{1}{2}$ feet deep, open at the top and covered with a screen of $2x\frac{1}{2}$-inch iron bars spaced 3 inches apart. The edges are strengthened by 3x3-inch angles, and there are hooks at the corners for raising and lowering the intake. The end of the intake pipe is flared out where it enters the intake, presumably to reduce the loss of head due to the entrance of the water into the pipe. A similar but smaller intake is used at Geneseo, N. Y., but at this place the outer portion of the submerged intake pipe is of the lap-welded type rather than cast-iron. Both works were designed by J. Nelson Tubbs, M. Am. Soc. C. E.

A 12-inch wrought-iron suction pipe about 2,000 feet long fitted with the Converse lock joint was laid through the ice at Escanaba, Mich., in 1888, under the direction of Mr. E. C. Cooke. The pipes were handled by three tripod derricks, the rearmost being carried to the front as fast as a joint was made. The joints were calked, and cross pieces placed over the pipe and supported on blocking. Each joint was hung by ropes from the cross piece over it. After a joint was calked, the ice was cut away for a pipe length, a man stationed at each rope, and, at a given signal, the pipe was lowered a short distance. When the entire pipe had been placed on the bottom in this manner, an operation requiring a gang of twenty men and costing $200, it was sunk about two feet into the sand by forcing out the material from below it by means of a water jet. Three men were required to run the portable pumping plant and operate the jet; the work lasted twelve days and cost about $75.

CHAPTER XVI.—THE CLARIFICATION AND PURIFICATION OF WATER.

Water supplies which are not clear are generally regarded as unsafe for domestic use, while those which are colorless and free from odor are considered satisfactory. This popular idea is wrong; the healthfulness of a supply depends on the presence or absence in it of the bacteria of disease. An examination of the source of the water will show if it is exposed to dangerous pollution, a chemical analysis will indicate if such pollution has taken place, and a bacterial examination will check certain deductions from the chemical analysis and indicate whether the water is fit for use in its natural state or should be purified. These investigations may show that a limpid stream is dangerous, and that a turbid one will furnish a satisfactory supply if it is clarified before use.

Turbidity and color in water are not the same; the former is due to the presence of silt, the latter to dissolved vegetable matter. It is so rarely necessary to resort to special means of removing color from water that a discussion of the subject is out of place here. Clarification by one method or another is frequently desirable and sometimes absolutely necessary, and its essential features deserve careful attention. Failure to recognize its importance has caused many mistakes in American water-works construction in the past.

The turbidity of water has recently been studied with great care by Messrs. George W. Fuller, Allen Hazen and other specialists. The silt which causes it is washed into the streams by heavy rains, and sometimes is composed of particles not more than 0.00001 inch in greatest diameter. Very muddy water may contain 0.1 per cent of its weight of these particles, which will clog any kind of a filter so rapidly as to make the filtration of the supply, on a large scale, impracticable until it has been at least

partially clarified. The turbidity is measured by determining the depth at which it is possible to see the point of a platinum wire thrust down vertically into the water. The reciprocal of this depth in inches has been arbitrarily assumed as the degree of turbidity. The observations should be made when there is good light, but the wire should be kept shaded if the sun is shining directly on it.

When turbid water is allowed to remain quiet, the heavier portions settle to the bottom within the first 24 hours, but after that period sedimentation proceeds so slowly that it is rarely expedient to provide for a longer period of subsidence. The fine particles of clay still remain in a large measure suspended in the water. While subsidence for 24 hours may reduce the total amount of suspended material 40 per cent. or so, it rarely has as much effect on the turbidity because the latter is due mainly to the very fine particles of clay still floating in the water. In the case of the Ohio River water at Cincinnati, Mr. Fuller found that the fine clay particles remaining after three days of subsidence settled very slowly, and the decrease in the amount of suspended material in any day after the third seldom exceeded 5 per cent. Hence the construction of large reservoirs solely to allow subsidence for several days is rarely advisable. There is generally no need for them except during the periods of flood, and it is usually possible at such times to accomplish their object by less expensive means.

The effect of a settling reservoir on the turbidity of water under certain important conditions is pointed out clearly by Mr. Allen Hazen in an article reviewed in "The Engineering Record" of March 25, 1899. He assumed a reservoir holding 24 hours' supply into which the water is pumped constantly at one end and from which it is drawn constantly at the other. The water is drawn from a stream which has a catchment area above the intake of such size and shape that the runoff from the area passes the intake in less than 24 hours. Under ordinary conditions the water is comparatively clear, and is unchanged by passing through the reservoir. When a storm occurs, the stream becomes turbid, and muddy water is pumped into the basin, but, owing to the time required for passing through it, the effluent remains clear for some hours. There is a gradual mixing, however, and long before the expiration of 24 hours, somewhat muddy water appears at the outlet. The turbidity in streams of this size rarely lasts more

than 24 hours, and at the expiration of that time the water in the reservoir is as muddy or muddier than the water flowing in the stream. Generally the improvement in the stream is several times as rapid as in the basin. The only time the latter is of use is during the first of the flood, and its value is then due mainly to its storage capacity. Under the assumed conditions the average quality of the water can be greatly improved by using the reservoir for storage rather than sedimentation. It should be kept full during clear-water periods and pumping to it should be stopped whenever the turbidity exceeds a certain limit, and the supply drawn from the reservoir until the turbidity again falls to the usual degree. As the stream becomes larger and the turbid periods longer, the size of the storage reservoir must be increased.

In case a reasonable amount of sedimentation does not clarify the water sufficiently, it is necessary to resort to filtration for further treatment, but it is highly important to have as little silt as practicable in the water applied to the filters. The latter continue the process of clarification by acting as strainers, by the adhesion of the suspended material to the grains of the filters, and by the sedimentation which occurs in them on account of the slow velocity with which the water passes downward. The last is particularly important in the case of water containing very fine suspended material, and may make it necessary to operate the filter at a much slower rate for clarification than is needed for satisfactory bacterial purification. In case sedimentation and these three processes of clarification are inadequate to accomplish the desired result, recourse must be had to the use of a coagulant, which will unite the minute particles of suspended material into masses of sufficient size to be stopped by the filter. Sulphate of alumina is the coagulant commonly employed for the purpose. The carbonate of lime dissolved in the water unites with it chemically and two new substances result; sulphate of calcium, which remains in solution, and hydrate of aluminum, which has the form of little flakes of jelly. It is the latter which collect the floating particles of clay and carry them to the filter. The success of coagulation in this manner depends on the presence in the water of enough dissolved lime to change the sulphate of alumina in the manner mentioned. Unfortunately, the amount of dissolved lime is generally least during floods, when it is most necessary, and, as Mr. Hazen has shown very forcibly, it may happen that serious trouble

will be experienced at such times, even with waters which ordinarily present no difficulties in this respect.

SAND OR ENGLISH FILTERS.

The usual purpose of filtration, at least of filtration through large beds of sand at low rates, is not so much the clarification as the purification of the water. The general nature of the process is readily understood. The raw water is clarified if necessary and then turned over the top of a layer of sand, which is supported by a bed of well-drained gravel. It passes through the sand and is clarified in its course by the three causes previously mentioned. A result of this clarification is the formation of a layer of sediment on the surface of the sand, which is itself a filter of much finer pores than the original layer of sand. The interstices are so small that the bacteria are caught in them and the water is thus purified as well as clarified. Various chemical changes also take place in the water, which are not definitely understood. As the sediment on the surface of the filter becomes thicker and thicker, the resistance it offers to the passage of water increases, and it is necessary to increase the depth of water over the sand in order to keep the rate of filtration uniform. Finally the resistance of this surface film becomes so great that it must be removed, which is accomplished by skimming off half an inch to an inch of the top of the sand bed. The sand which is removed can be washed and used again, or it can be thrown away and a fresh supply obtained; generally it is cheaper to wash the sand. In order to clean the filter it is necessary to throw it out of service for a time, and it is therefore desirable to have a number of filter beds, so that the temporary disuse of one of them will not interfere with the supply of filtered water to the community.

Inasmuch as the efficacy of filtration depends very largely on the character of the sand used in the beds, it is important to consider this subject in detail. The sand must be clean, and should consist mainly of sharp quartz grains. The remaining particles may be hard silicates, although pure quartz sand is desirable. Its physical properties are commonly classified by the methods established several years ago by the specialists of the Lawrence Experiment Station of the Massachusetts State Board of Health, particularly by Mr. Allen Hazen, whose book on the filtration of public water supplies should be purchased by the water board of every

community where filtration is proposed or practiced. The investigations of these engineers have shown that the value of any sand for filtering depends very largely on the smaller particles, and on the uniformity of size of the grains. The first relation is expressed by stating the effective size of the given sand, which is arbitrarily assumed to be the size of grain which is exceeded by 90 per cent. by weight of all the particles. The second relation is expressed by stating the uniformity co-efficient, the ratio of the size of the grain which has 60 per cent. of the sample finer than itself to the size which has 10 per cent. finer than itself. The latter size, it will be observed, is the effective size. The various sizes are determined by sieves, and are expressed in millimeters, as the metric system of weights and measures is now universal in all scientific work, not only on account of the labor it saves but also because it is the common system of scientists of all nations. A millimeter is 0.03937 inch.

The sieves used for separating the particles are made of brass wire and are shaken by hand or by an apparatus illustrated in "The Engineering Record" of January 23, 1897. Mr. James H. Fuertes uses a set of tin cups fitting end to end, each cup having a screen soldered in the bottom. A sample of sand is placed in the top cup and the entire nest shaken by hand. A series which has been found useful has approximately 2, 4, 6, 10, 20, 40, 70, 100 and 200 meshes to the inch, although the exact number is of no importance. The real size of each screen is determined by experiment. A thousand or so of the last particles to pass the sieve, except in the case of those of the larger meshes, are caught, counted and weighed. The total weight of these grains divided by the total number gives the average weight of a particle. The specific gravity of the sand, which runs from 2.6 to 2.7 in material suitable for a filter, should be determined. The size of the screen is then assumed to be that of the diameter of a sphere of the weight and specific gravity of the average of these particles which passed it last. The volume of the sphere is the weight divided by the specific gravity, and the volume multiplied by 1.91 gives the cube of the diameter. If the weight is expressed in milligrams, the diameter will be expressed in millimeters. The size of each screen is determined in this manner, and when this has once been done it is an easy matter to find the effective size and uniformity co-efficient of any sand. A sample is carefully weighed and passed

through one sieve after another until one is found which will pass only 10 per cent. of the total quantity. The size of this sieve is the effective size of the sand. In the same way a sieve should be found which will pass 60 per cent. of the sample. The ratio of this size to the effective size is the uniformity co-efficient.

Although these terms are employed in a largely arbitrary manner, yet their value is evident when a study is made of the utility of various sands for filtering purposes. It has been determined for example, that the smaller the effective size of the sand the greater the bacterial efficiency of the filter, tests at Lawrence giving results shown in the accompanying table. While the size of grains is thus found to affect the degree of purification, it was learned at the same time that the frequency with which it is necessary to scrape a filter during the course of a year depends upon the effective size of the sand. These results are given in the same table. In each case the maximum loss of head before scraping was 70 inches.

The Influence of Size of Sand on Filtration.

(From Rept. Mass. State Board of Health, 1893.)

Effec. Size.	Unif. Coef.	Depth. Ins.	Rate of Filtra. Gals. per Acre Daily.	Bacterial Efficiency. Per Cent.	Total Effluent bet. scrapings. Gallons.
0.09	2.1	45	1,580,000	99.978	14,000,000
0.14	2.2	60	2,160,000	99.964	49,000,000
0.20	1.6	55	2,420,000	99.990	56,000,000*
0.26	3.7	60	4,848,000	99.901	57,000,000
0.29	2.7	60	4,940,000	99.917	70,000,000
0.38	3.5	60	3,800,000	99.840	79,000,000

* This figure is taken from the Report for 1892.

These figures show that there is a limit beyond which it is not practical to decrease the size of the sand grains; this limit is different in filters which must treat river water containing silt and those for clear lake water. The long experience of the managers of the London water companies has led them to use sands which, when washed, vary in effective size between 0.25 and 0.36 millimeter and have a uniformity co-efficient of 1.8 to 2.4. These filters are used to purify water which has considerable turbidity at times, but is subject to sedimentation for several days before filtration. The Hamburg filters use a washed sand of an effective size of about 0.3? millimeter, with a uniformity co-efficient of about 2.3. These are also used with river water. The Mueggel Lake filters of the Berlin works use sand of 0.35 millimeter effective size, which has a uniformity co-efficient of 1.8. The celebrated Zurich filters use

a sand of 0.3 millimeter effective size with a uniformity co-efficient of 3.1, which is very high. As a general rule, the sand selected for a filter should have an effective size between 0.2 and 0.35 millimeter, and a uniformity co-efficient under 3.

The thickness of the sand bed should never fall below 12 inches, and 16 inches is a much better limit. The thickness when new may be from 30 to 40 inches. Experiments made by the Massachusetts State Board of Health in 1896 show that the effluent from thick beds is better than that from thin ones, other conditions being the same. It is customary to store the sand scraped from the filters from time to time until the depth remaining is the minimum. Then the sand is washed, and the bed brought back to its original level.

It is essential to have each filter bed in a water-tight basin. Sometimes puddle covered by a pavement is used as lining for this purpose, and sometimes concrete, the preference being for the latter. Small filters enable a better control to be exercised over the filtration, and permit cleaning or renewal without interfering to any great extent with the supply; they are also of decided advantage if it is necessary to cover the filters as a protection against freezing. Large beds are less expensive to construct on account of the fewer walls and embankments needed with them. From one-half to one acre will probably be found the limiting sizes of each bed under most conditions, although in small works it may be desirable to use beds as small as a quarter of an acre each in order to provide for the exigences of cleaning and slower filtration during cold weather. If the filters are to be built at a place where the January temperature is below the freezing point it will probably prove advisable to cover them. This can be done in several ways, but that adopted for the beds at Somersworth, N. H., by Mr. William Wheeler, M. Am. Soc. C. E., and illustrated in "The Engineering Record" of August 27, 1898, will probably prove suitable for many places. A discussion of the principles on which such a roof should be designed is outside the scope of this book; they are stated concisely and clearly in Prof. I. O. Baker's "Treatise on Masonry Construction."

The sand must be supported on some coarse material in order that water passing through it may run off as freely as possible. Formerly, the support was a series of layers of coarse gravel and sand, growing finer with its distance above the floor of the basin.

It has been found by experience that this complication of strata is an unnecessary expense. It is probable that 6 inches of coarse gravel and three to five 2-inch layers of material uniformly placed and decreasing in size to coarse sand in the top layer, will prove amply sufficient to support the bed of sand and allow the passage of water. Care must be taken that the stones or grains of each layer are fairly uniform in size, and that their relative sizes are such that the material of any one layer will not be driven down into that below.

The gravel contains a network of underdrains through which the filtered water is drawn off. Tile drains are generally employed for the lateral branches and these discharge into a main drain with sides of dry brickwork and a cover of stone slabs, allowing water to enter it freely. All the drains should be so arranged that the sand has a uniform thickness at every part of the bed. Mr. Hazen has suggested that the limiting areas round tile drains should be required to drain are as follows: 4-inch, 290 square feet; 6-inch, 750 square feet; 8-inch, 1,530 square feet; 10-inch, 2,780 square feet; 12-inch, 4,400 square feet. Larger drains should have an area at least 1-6,000 of the area drained.

The apparatus for applying water to the filter beds and for drawing it off vary widely. Some means must be provided for regulating the filtration head so that the bed yields a definite quantity of water all the time it is in service. For small beds the best method of accomplishing this will probably lie in maintaining a fixed depth of water over the sand and varying the elevation of discharge of the filtered water. If this elevation is nearly that of the water level over the bed the discharge will be small, and if the effluent is drawn from a lower point the quantity will be greater, provided the bed has not become clogged meanwhile. A good idea of this system of regulation is furnished by Figures 34 and 35, giving a plan and details of a plant with a nominal daily capacity of 1,500,000 gallons recently built at Berwyn, Pa., from the designs of Mr. J. W. Ledoux, M. Am. Soc. C. E.

There are three beds of 7,500 square feet each in a basin with rubble masonry side and partition walls, and a bottom of 8 inches of puddle covered by 3 inches of concrete. The sand has a thickness of 30 inches, an effective size of 0.25 millimeter and a uniformity co-efficient of 1.82. It rests on gravel 6 inches thick and ranging in size from particles the size of a pea at the top to 1½-inch pebbles

at the bottom. The main drains are vitrified sewer pipes with cement joints laid in a trench bringing their top to the level of the bottom of the basin in other places. These drains terminate in cast-iron pipe running through the walls. The laterals are 6 feet apart and formed of ordinary 4-inch tile in 12-inch lengths which were placed end to end without other joint. They are also

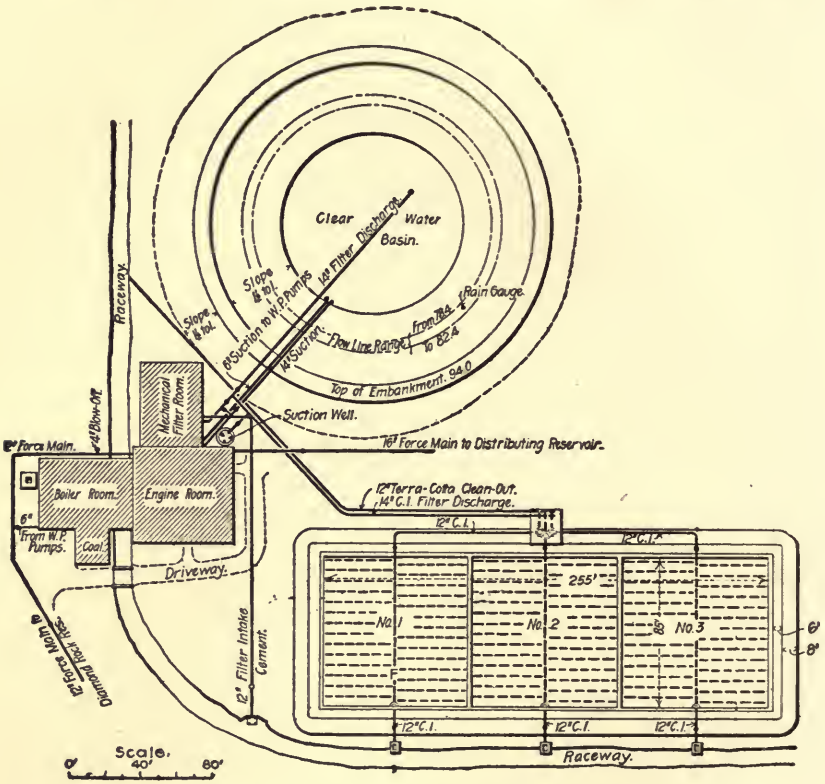


FIGURE 34.—PLAN OF THE BERWYN FILTERS.

placed in depressions in the concrete bottom and are covered with 4 inches of gravel.

The water enters each bed through an intake on a raceway and a 12-inch cast-iron pipe running under the embankment to an inlet chamber. This is merely a semi-circular wall 9 inches thick and as high as the bed of sand. The water rises in it freely and mingles freely with that over the sand bed without disturbing the

latter in any way. A wire screen is placed over the intake at the raceway to keep refuse from reaching the filter. It is not unusual to provide an overflow chamber of some sort or else use a

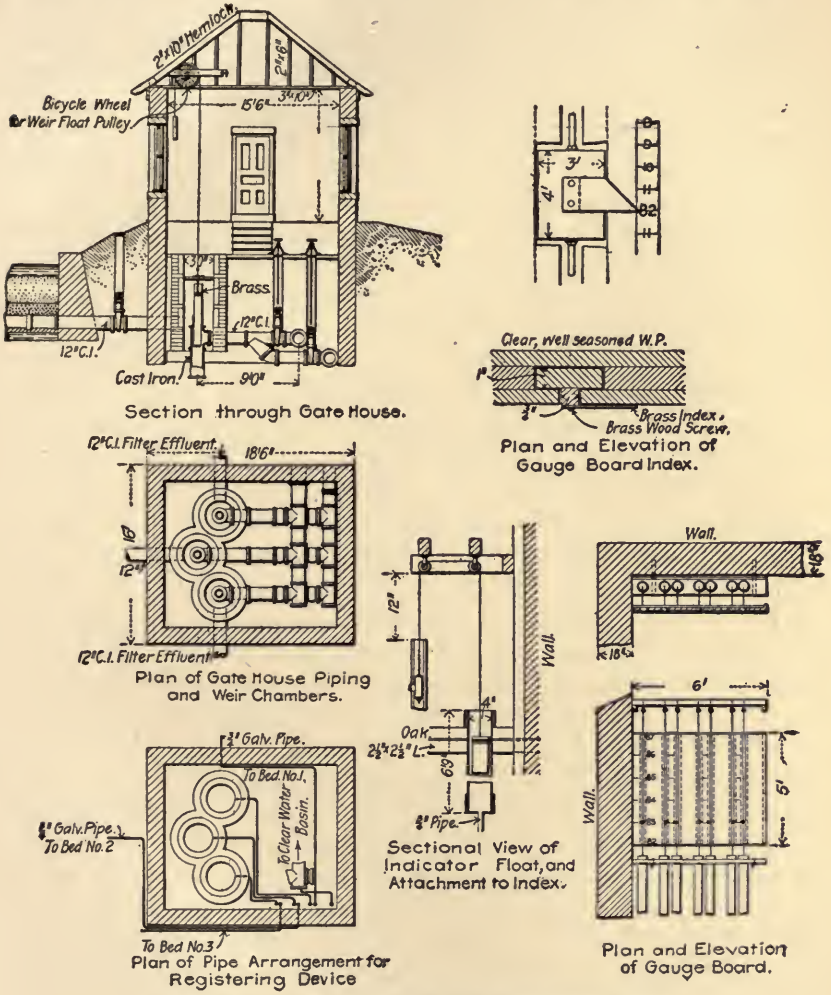


FIGURE 35.—GATE HOUSE OF THE BERWYN FILTERS.

balanced float valve controlling the flow of water through the inlet pipe, in order to prevent the water on the beds exceeding a certain height.

The provision for maintaining a uniform rate of percolation

through the filter beds is described by Mr. Ledoux substantially as follows: The regulating apparatus consists of a brass tube open at both ends and hanging to a float which rises and falls with the water level in the effluent chamber. The top of the tube may be considered a submerged circular orifice or weir, which is kept at a constant although adjustable distance from the float. The float and sliding tube are counterweighted as shown in Figure 35. As the sand bed becomes clogged by sediment, the water level in the effluent chamber will sink and lower the float and weir, thus increasing the effective head. An indicator board is provided in the gate-house. It has seven indicators, one for each filter bed, one for each effluent chamber in the gate-house, and one for the clear-water basin. The difference in level between the water on a bed and in the corresponding effluent chamber, which indicates the loss of head, is shown at a glance. The indicators slide in vertical grooves and are attached to No. 26 copper wires, which run over brass pulleys. Attached to the other end of the wires are floats made of 3-inch nipples capped at each end. These floats work in pipes having half-inch pipe connections with the water on the filter beds, in the effluent chambers and in the clear-water basin.

The cost of this entire filter system was $18,536; the most expensive single item was $4,420 for 2,697 tons of sand from Gloucester, N. J., and the next was $2,927 for 528 yards of stone masonry.

There are various methods of washing sand, those mainly employed involving the use of hoppers of the form shown in "The Engineering Record" of October 19, 1895, or drums of the type illustrated in the issue of the same paper for October 29, 1898. In the case of small works it will probably prove as satisfactory to place the sand on the slightly sloping, watertight bottom of an enclosure having a low weir at its lowest side, and wash it with a hose until clear water runs over the weir.

MECHANICAL FILTERS.

So far in the discussion of filtration it has been assumed that the filters were of the English type, in which the sand is scraped off when the flow through it falls below a certain rate. In recent years, a different type of apparatus, called the American or mechanical filter, has been developed, and is rapidly replacing the English beds for many situations in the estimation of engineers. Its characteristics are described so clearly in Mr. George W. Ful-

ler's invaluable report on the Louisville filtration experiments, that an extract from this volume is printed here:

"This type of filters is the outgrowth of schemes to purify water for industrial and manufacturing purposes. Its development up to this time has been tentative to a marked degree, and has been in the hands of several competing business corporations. In 1883 it first attracted the attention of those connected with public water supplies. At that time it consisted essentially of a large circular tank in which there was a layer of sand supported by a perforated bottom. Its chief characteristic, other than small size, in distinction from sand filters, was the fact that the sand layer was cleansed of the accumulated materials removed from the river water by forcing water under pressure up through the layer of sand. In this respect it resembled the filters constructed in 1856 at Tours, in France.

"Patents were taken out in 1884 to cover a modification which consisted of the application of alum, a salt of iron, or other similar coagulating chemical, to the water just before it passed through the layer of sand. The custom of applying alum to coagulate water, in order to facilitate the removal of foreign matter, has been practiced in various ways for many centuries in different parts of the world, and the description of it in scientific literature began about 70 years ago. The apparent object of the application of chemicals under the stated conditions are understood to be a reduction in the cost of treatment, by doing away with subsidence basins and by diminution of the area of filtering surface.

"This type of filter was first employed in the treatment of a public water supply at Somerville, N. J., in 1885. Since that time many towns and small cities have adopted systems of this general type. At present it is said that over 100 town and municipal plants are in operation, but among this number there are none for large cities.

"In the last ten years many modifications have been introduced by the several competing companies. These modifications, more or less protected by patents, relate for the most part to devices for supporting the sand layer at the bottom; the introduction of filtered water under pressure below the sand filter, to enable the filter to be cleaned by a reverse flow of water; and of agitating devices to stir the sand during washing, and thus aid the cleansing process. In the present filters of the several companies the coagu-

lating chemicals are applied at points differently located with reference to the sand layer, and with varying provisions to secure not only more complete coagulation but also to effect a removal of some suspended matter before the water is filtered. To this general account of the American filters it may be added that a majority of them are gravity filters—where the water flows by gravity through a sand layer placed in an open tank. In some cases, however, pressure filters are used. The pressure filters, in addition to customary devices, consist of a sand layer placed in a closed compartment, so that water can be forced through the filter under pressure, thereby avoiding, it is claimed, additional pumping under some conditions.

"Compared with the English filters, the American filters at present show the following principal differences. 1. The American filters are aided by the application to the water of a coagulating chemical, which makes it possible to filter through sand at a much more rapid rate, and thereby the necessary area of filter is much reduced. 2. The American filters are cleaned by passing a stream of water upward through the sand, with or without accompanying agitation, rather than by scraping off the surface layers, as in the case of the English filters.

"There are of course many other features of difference, such, for example, as the strainers at the bottom, to hold back the sand and at the same time furnish an exit for the filtered water; but the two points stated above are the principal differences."

It is natural to ask why the first distinctive feature of mechanical filters, the use of a coagulant, is not adopted with the English type of sand filters. The answer is probably to be found in the fact that such filters have not been used on a water of sufficient turbidity to make them necessary. There is no reason why alum should not be so employed, and the subject was carefully studied by Mr. Fuller in his investigation of methods of purifying the Ohio River water for use in Cincinnati. This water contains at times such a large quantity of fine clay particles that it is impracticable to filter it through sand beds, even after several days' sedimentation. He investigated two methods of treatment:

1. Applying the chemical to the plain subsided water when its condition demanded it, and then allowing the coagulated portions in suspension to subside in a relatively small basin interposed between the plain subsiding reservoirs and the filters.

2. Applying the chemical to the river water, when required, be-

fore the water entered the plain subsiding reservoirs, and allowing the supplementary clarification to take place in the main subsiding reservoirs, thus dispensing with the small intermediate basin.

The second plan proved preferable for economical reasons. After the water is uniformly prepared for filtration, the English type of filters can complete the clarification and purification in a satisfactory manner and with a smaller area of filters than is conventionally considered to be necessary. The methods of applying the coagulant are much more satisfactory with mechanical filters, however, and in the carefully prepared estimates of the total cost of filtering the Ohio River water by both systems of filters, the mechanical plant had a marked advantage.

The details of these American types of filters are so well shown in the publications of the companies making them that it is unnecessary to describe them. The mechanical filter has successfully lived down the attempts made to discredit it, and it has been definitely proved able to filter water satisfactorily when properly handled at rates of over 100,000,000 gallons per acre daily. Intelligent management is essential for good results. Whether the mechanical or sand filter should be employed in a given case, is to be settled by a careful study of the local conditions, cost of plant, operating expenses and the durability of the various parts.

CHAPTER XVII.—THE PIPE SYSTEM.

The calculation of the size of a pipe needed to furnish a certain quantity of water under given conditions is readily accomplished if approximate results will serve the purpose. If accurate computations are needed, it is allowable to indulge in some complicated algebraical equations if the engineer's mind is made any easier by so doing, but the results do not appear much if any more reliable. The writer believes that, in the light of present knowledge, the pipe computer devised by Mr. William Cox affords the readiest means of solving the problems presented by the flow of water in pipes. For all the purposes of small water-works, however, the following methods will answer.

If water flows from a reservoir through a straight pipe on a uniform grade to an outlet, the amount of the discharge depends on the force which causes the water to flow. This force is measured by the difference in elevation between the water levels in the reservoir and the outlet, which is called the total head, less the entry head which is spent in forcing the water into the end of the pipe, less the friction head spent in overcoming the resistance offered by the interior of the pipe to the passage of the water and in the churning and swirling of the water itself. This difference is the velocity head, and is the height through which the water would have to fall freely without friction for it to acquire the velocity it has in the pipes. It is evident that if a given quantity of water has to be taken through a conduit from one place to another, a small pipe in which the frictional resistance is comparatively great will make it necessary to have a larger total head than a pipe of greater diameter. If there is ample head then a small pipe may well be used, but it often happens that the difference in elevation between the inlet and outlet of the pipe is so slight that only a large pipe opposing little resistance to flow can be used.

The entry head is trifling compared with the friction head, in the case of water-works pipes, and the problem of designing a water

main practically hinges on the friction head. This may be estimated by means of the following equation:

Fric. head × Diam. = Coef. × Length × Square of Velocity.

In this equation the friction head is expressed in feet, the diameter of the pipe in feet, the length in feet and the velocity in feet per second. The coefficients depend on the diameter of the pipes, and are as follows:

Diam., Ins.	4	6	8	10	12
Coefficient	0.00038	0.00036	0.00034	0.00033	0.00033

The coefficient for any other diameter can be quickly calculated from the formula

$$\text{Coefficient} = \left(0.019892 + \frac{0.001666}{\text{Diameter}}\right) \div 64.34.$$

The diameter should be expressed in feet as before. The friction head found by this method is for new pipes. It becomes greater the longer the pipe is in use, and Mr. Edmund B. Weston, M. Am. Soc. C. E., advises increasing the amount for clean pipes by the use of the following multipliers:

Age of Pipes, years...	10	15	20	25	30	35
Multipliers	1.31	1.47	1.63	1.78	1.94	2.11

The use of the formula may be seen by solving a simple problem, such as a determination of the smallest size of pipe to deliver 750,000 gallons per day which it is safe to lay on a uniform grade between points 15,000 feet apart and differing 125 feet in elevation. The maximum natural grade per 1,000 feet is $125 \div 15 = 8.33$ feet. If it is necessary for the delivery of 750,000 gallons to remain constant for 20 years, the pipe must be made of greater present capacity in order to allow for subsequent tuberculation and clogging. Hence, from the preceding table of multipliers, the friction head used in making the computations must be 1.63 times that for clean pipes. A discharge of 750,000 gallons in 24 hours is equivalent to 521 gallons per minute. An inspection of Figure 36 shows that a 6-inch pipe, even when new, offers too much friction to permit of its use under the circumstances. With an 8-inch pipe, the friction head per 1,000 feet of clean pipe is seen to be about 5.85 feet. If this is increased by using the multiplier 1.63, the resulting product is 9.53 feet. Inasmuch as the natural grade between the ends of the pipe is only 8.33 feet it is evident that the 8-inch pipe will probably prove of insufficient capacity to deliver the needed volume of water before the end of the 20-year period. The friction

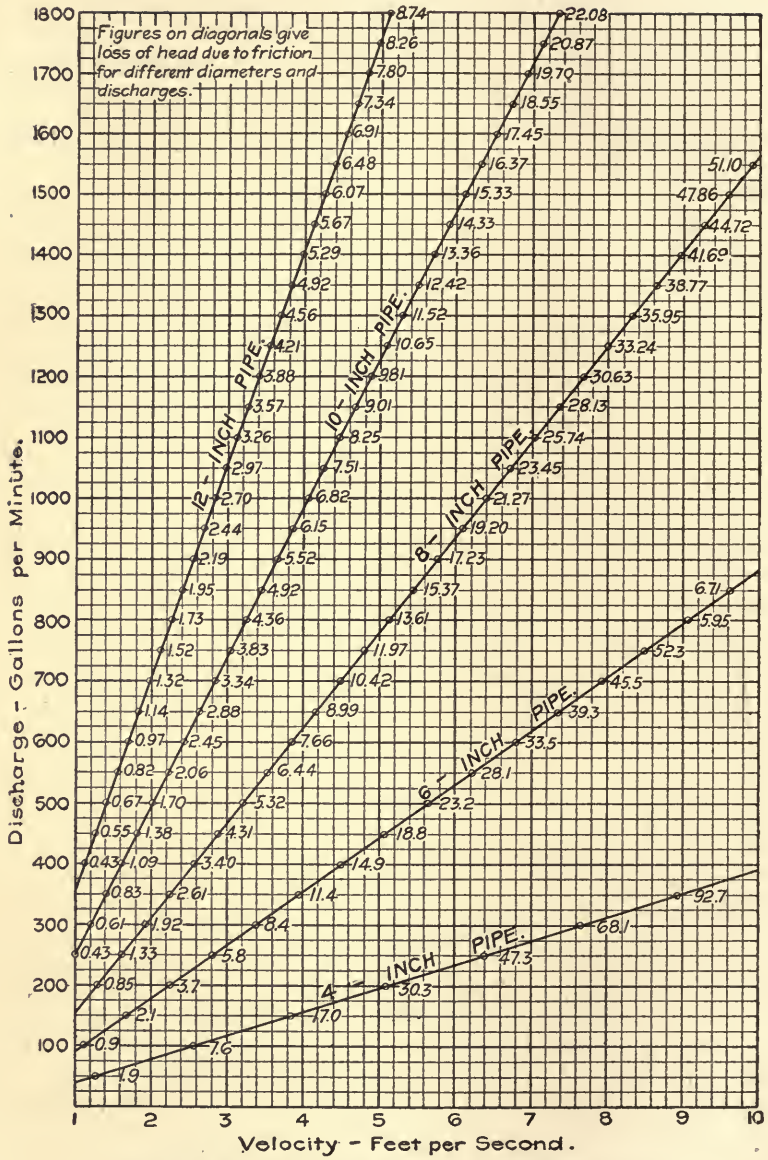


FIGURE 36.—DISCHARGE OF PIPES 1,000 FEET LONG.

head of a new 10-inch pipe discharging 521 gallons per minute is about 1.84 feet, and when the main has been in service 20 years it is fair to assume it will not be more than $1.84 \times 1.63 = 3$ feet. As the available head is 8.33 feet, this 10-inch pipe will discharge more water than is wanted. It will not cost much more than one 8 inches in diameter, however, and will deliver the water under a good head at the outlet end of the pipe, which is frequently an important matter.

It rarely happens that there is a uniform slope of the ground from one end of the pipe to the other, and on this account it is necessary to investigate a little more carefully what may take place if the profile of the main is wavy. In Figure 37 A represents the reservoir from which water is taken, and B the basin into which it is discharged. There is an easy slope from A two-thirds of the distance toward B, and then the ground falls away more rapidly. If a pipe of uniform diameter is laid a few feet below the surface, it will not discharge the amount of water which might be expected to pass through a main of that size running between points of the given difference in elevation.

The explanation is to be sought in the presence of the vertical bend at C. Here the grade of the two portions of the pipe changes. The result may be understood most easily from the preceding formula for friction head by substituting "slope" for "head" divided by "length," and for "velocity" the "discharge" divided by the "area" of the pipe. The formula thus becomes:

$$\text{Quantity} = \text{Area} \sqrt{(\text{Diam.} \times \text{Slope} \div \text{Coef.})}$$

It is evident from this equation that with a pipe of constant diameter the part which slopes the more will have a greater carrying capacity, other things being equal. In this particular instance shown in Figure 37, the slope from A to C, is $I \div M$, and that from C to B is $(H - I) \div N$. The section C B of the pipe will be but partly full and the section A C will deliver only the amount of water which is due to the difference in elevation I. Since the section C B acts merely as a trough, it may better be made of smaller diameter; in other words, the two sections should be calculated independently, by means of the formula just given, to discharge the desired quantity of water.

It is now easy to understand what is meant by the term hydraulic gradient. In Figure 38, A is the source of the supply and B is the point of discharge. At C there is a ridge of land projecting

above the straight line drawn between the water levels at the ends of the pipe, and at this point the pipe rises above the straight line. From what has already been said it follows that if there were two pipe lines of the same diameter from A to C, one following the profile of the ground and the other laid along the straight line, the former would not discharge as much as the latter. The straight line is called the hydraulic gradient, and no pipe line of uniform diameter should rise above it. In case it is impracticable to keep below it, the main should be regarded as a succession of pipe lines and each of these calculated, independently of the others, to discharge the desired quantity of water. The frequently made statement that a pipe should not rise above the hydraulic gradient is ridiculous unless restricted to the case of a main of uniform diameter from end to end. The importance of obtaining a fair profile of the country between the source of a water supply and the point of its discharge is evident from this brief discussion;

FIGURE 37.

FIGURE 38.

without such a profile it is impracticable to ascertain whether the main is above or below the hydraulic gradient.

In case the water must be pumped from a lower to a higher elevation, precisely the same method of reasoning must be followed. There is the head due to the difference in elevation between the pumping station and the point of delivery, and the head due to the friction of the force main; the sum of the two is the head against which the pump operates. The friction head is calculated precisely as in the case of a gravity main, and is less in a large than a small pipe. The force main, if of small diameter, requires more powerful pumping machinery than is the case when the diameter is larger. The power of the pump and the diameter of the force main should, therefore, be determined together. The cost and life of pipes of different sizes must be compared with the cost, operating expenses and life of pumping plants of powers corresponding to the different pipes. The set of corresponding figures which has the smallest sum is the one to

be selected. A complete analysis of such a problem in connection with works of larger capacity than those discussed in this book will be found in "The Engineering Record" of February 19, 1898.

It has been assumed so far in this chapter that the problem is to compute the size of a pipe of given length to discharge a certain quantity of water. It is often desirable to determine how much water a pipe already laid may be expected to yield. In this case the first thing to do it to determine whether the pipe rises above the hydraulic gradient at any place; if it does not the total length and fall of the pipe may be used in estimating the discharge, and if it does the section of pipe having the flattest grade will be the one to employ in making the computations. Express this slope in feet fall per 1,000 feet of distance. Multiply the coefficient for new pipe of the given diameter by the multiplier corresponding to the number of years the pipe has been in service. Then, having the area, diameter and slope of the pipe, compute the quantity of water it will discharge.

If the main is a succession of pipes of different diameters, the total friction for a given rate of discharge is found by adding together the friction heads in each section. If the problem is to find the discharge of such a compound main, having given the total head, and length and head of each section, it is necessary to resort to a roundabout method of figuring. Assume some discharge, and then compute what the total friction head would be in the given pipe when delivering such an amount of water. Then divide the really available head by this calculated head, and then multiply the assumed discharge by the square root of the quotient to obtain the actual discharge. The calculation can be made on the slide rule with a single setting, and the result will be sufficiently accurate for practical purposes if there is not much difference between the assumed and final discharge. If there is much difference it may be well to use the final discharge of the first computation as the assumed discharge of a second. These and many other problems are explained clearly in Mr. Freeman C. Coffin's "Graphical Solution of Hydraulic Problems," an invaluable book to all engineers engaged in hydraulic work.

There remains for consideration the problem of a complex main, Figure 39, in which the water reaches a given point, E, by two routes, one directly along A B E and one along A B C D, a problem given in Mr. Coffin's book. It is desired to determine

the loss of head between A and E due to a draft of 1,000 gallons a minute at E. The loss from A to B will be 6.7 feet by Figure 36. From B the flow divides. It is self-evident from the conditions that the loss of head along B E is the same as that along B C D E, otherwise there would be two pressures at E, which is impossible. It is necessary to assume some loss, and as it is desirable to make as close a guess to the truth as possible, it will be well to refer again to Figure 39. If three-fourths of the water went by the line B E, it is evident from Figure 36 that the friction head would be about 7 2-3 feet, while the friction head in B C D E for the remaining fourth of the flow would be about 5 1-3 feet. As an assumption of 6 feet is, therefore, probably somewhere near the truth, it will be employed in computing the discharge through the two pipes, which will be assumed to be new. A loss of head of 6 feet on 2,000 feet of 10-inch pipe corresponds, according to Figure 36, to a discharge of about 660 gallons, and an equal loss of head along 4,000 feet of 8-inch pipe corresponds to a discharge of about

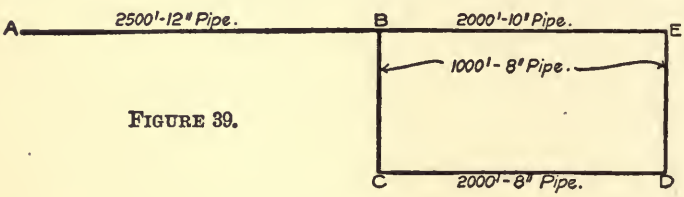


FIGURE 39.

265 gallons. The sum is 925 gallons. It is now necessary to divide the desired discharge by that calculated for the assumed friction head, and multiply the square of the quotient by the assumed friction head. Hence,

$$6\ (1{,}000 \div 925)^2 = 7.0 = \text{Friction head.}$$

This head added to 6.7 feet from A to B gives 13.7 feet as the total head required to deliver 1,000 gallons of water from A to E over the complex main. As a matter of fact, this result is probably about 0.7 foot too small, but to obtain more accurate results it will be necessary to refer to the tables and diagrams given in Mr. Coffin's book, or indulge in some elaborate calculations. The method outlined will answer, however, to give the approximate figures generally desired.

The method of laying pipes and conducting other construction work connected with a water plant, is described so fully in the excellent book by Mr. William R. Billings entitled "Some Details

of Water-Works' Construction," published by "The Engineering Record" at two dollars, that it is unnecessary to devote space to the subject here.

DATA CONCERNING PIPE AND ACCESSORIES.

In the preceding portions of this chapter it has been assumed that cast-iron pipes were used. As a matter of fact riveted pipe has been employed to a considerable extent for small works in some States where the freight charges for cast-iron pipe have been so high as to prevent its use. The common spiral riveted pipe, manufactured for regular commercial sale, is generally assumed to have a friction loss 15 to 20 per cent. greater than that of good cast-iron pipe of the same diameter, at least, in the small sizes. Riveted pipe of special construction, such as that used by Mr. A. L. Adams for a 14-inch and 16-inch conduit for the Astoria, Ore., water-works, does not offer so much resistance to flow.

The steel of this conduit was required to have an ultimate strength of 58,000 to 65,000 pounds, an elastic limit of 30,000 pounds, 25 per cent. elongation in 8 inches, and be able to withstand, without fracture, being bent cold and hammered flat. The sheets came in 4-foot lengths, and were made up in alternate large and small courses, the last, a small one, being made slightly conical and expanded to the diameter of a large course at the end of the pipe. Straight seams were double riveted and round seams single riveted, the holes being punched from the sides coming together in the lap. The pipe was required to be tight before dipping and every length was tested in that condition. Drops of water were not regarded as leakage. The pipes which passed the test were coated with hot asphalt by dipping them in tanks of this material. The joints were made by means of sleeves of $\frac{3}{8}$-inch welded iron 6 inches wide, with a lead space of about $\frac{3}{8}$-inch. A reinforcing thimble of No. 8 steel 8 inches in width was inserted half its width at the shop and riveted in one end of each pipe. The crack at the junction of the two pipe ends was filled with oakum while the annular space was filled with lead only. A further account of this pipe will be found in the "Transactions" of the American Society of Civil Engineers, Volume xxxvi.

Riveted pipe are often made slightly conical, the small end of one length fitting into the large end of the next. The large end is laid down hill, the pipes are driven together by wood mauls, and when the main is finished fine dirt or clay is thrown into the

upper end. This is washed into the joints and makes them water-tight.

Wood stave pipe is also coming into extensive use in the West. It is made by banding together carefully milled staves, and the relation between the size and spacing of the bands, the pressure to be sustained, the thickness of the wood and other factors is a subject far too complicated for discussion in this place. The principles governing these matters were explained at length in "The Engineering Record" of October 8, 1898. The friction loss in these pipes is believed to be somewhat less than in cast-iron pipes. The Wyckoff wood pipe, which has been used in a number of works, is made from creosoted staves held by a spiral steel band, and has given good satisfaction wherever it has been used, so far as the writer has been able to learn.

The cast-iron pipe for water-works should be cast vertically with the bell down in order to give every possible advantage to this end. It is customary to require the metal to be free from cinder iron or other inferior admixtures, and to be remelted in a cupola or air furnace before the cast. It ought to have an even grain and be drilled and cut satisfactorily. Specimen bars of a good metal for the purpose, 26 inches long, 2 inches wide and 1 inch thick, should sustain a load of 1,900 pounds at their center when placed flatwise on supports 24 inches apart; the deflection should be not less than 0.3 inch when the bars are broken. The tensile strength of the metal should not be less than 16,000 pounds. The pipe should not be taken from the flasks until sufficient time has passed to prevent unequal contraction of the metal on account of its premature exposure to the air. All pipes and special castings should be smooth, free from lumps, scales, blisters, sand holes and other imperfections. When the pipe has been thoroughly cleaned it should be inspected to see that the axis is straight, the metal sound and of uniform thickness, and the hubs and spigots of proper shape. It should then be heated to about 300 degrees Fahrenheit and dipped for at least 5 minutes in a bath of the material known as the Dr. R. Angus Smith pipe coating. After it has been removed from the bath and cooled it should then be subjected to a hydrostatic pressure of 150 to 300 pounds, according to its proposed use, and tapped with a light hammer to see that it is sound and perfect when under such a pressure. The inspection of the pipe at the foundry is usually done by an in-

spection company, of which several have cards in "The Engineering Record."

For small plants it is entirely unnecessary to require the manufacturer to cast special cast numbers and similar lettering on the pipe; sufficient letters to identify the foundry are all that are needed, for if the pipe proves to be poor when it is delivered it is just as easy to send it back if it bears only the trade mark of the foundry as if it is decorated with the names of all the town officials. The form of the spigots and hubs should be that generally adopted by the large cities in the vicinity. Some engineers have a sort of mania for devising special forms, which only make unnecessary trouble for the foundrymen; there are probably a dozen or more equally good ones to select from already, and many more which should never have gone farther than the drafting board.

The greatest water-works system in course of construction while this chapter is being written is the Metropolitan works of Massachusetts, for the supply of the Boston district of that commonwealth. The thickness of the pipes for those works is calculated by the following formula:

$$t = \frac{(p+q)\,r}{3300} + 0.25.$$

In this expression t is the thickness of the shell of the pipe in inches, p the static pressure in pounds per square inch, q the pres-

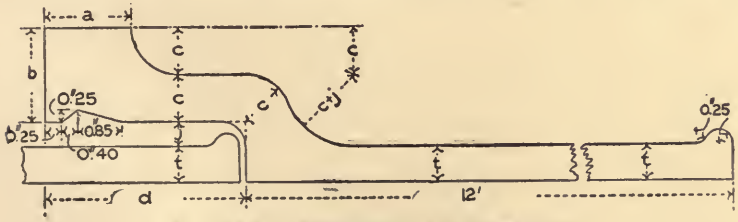


FIGURE 40.—METROPOLITAN PIPE JOINT.

sure in pounds allowed for water ram, and r the internal radius of the pipe in inches. The denominator 3,300 is one-fifth of the assumed tensile strength of cast-iron, 16,500 pounds per square inch. The value of q for pipes of 10 inches diameter or less is assumed to be 120, 110 for 12 or 14-inch pipes, 100 for 16-inch and a gradually decreasing number as the size increases. Designs were made for five classes of pipes; Class A for pressures up to 50

pounds per square inch, Class B up to 65 pounds, Class C up to 87 pounds, Class D up to 109 pounds and Class E up to 130 pounds. In the accompanying table of weights the iron is assumed to weigh 0.2604 pound per cubic inch, and the lead joints to be 2 inches deep for pipes of 14 inches diameter and less and 2¼ inches for larger sizes.

Weights of Straight Pipes, Metropolitan Water-Works. All dimensions are in feet and refer to Figure 40. The weight of the pipe is in pounds per foot and also per length, and the weight of the lead is for a single joint.

Diameter.	Class.	Dimensions in Inches.						Weight in Pounds.		
		a	b	c	d	t	j	Pipe Length.	Per Foot.	Lead Joint.
4	D	1.50	1.30	0.65	3.00	0.40	0.40	230	17.3	7.00
4	E	1.50	1.30	0.65	3.00	0.45	0.40	255	19.7	7.00
6	D	1.50	1.40	0.70	3.00	0.46	0.40	380	29.2	9.75
6	E	1.50	1.40	0.70	3.00	0.50	0.40	415	31.9	9.75
8	D	1.50	1.50	0.75	3.50	0.52	0.40	565	43.5	12.50
8	E	1.50	1.50	0.75	3.50	0.55	0.40	600	46.2	12.50
10	D	1.50	1.50	0.75	3.50	0.60	0.40	800	62.4	15.25
10	E	1.50	1.50	0.75	3.50	0.63	0.40	840	65.7	15.25
12	B	1.50	1.60	0.80	3.50	0.57	0.40	910	70.3	17.75
12	C	1.50	1.60	0.80	3.50	0.61	0.40	970	75.5	17.75
12	D	1.50	1.60	0.80	3.50	0.65	0.40	1,030	80.7	18.00
12	E	1.50	1.60	0.80	3.50	0.69	0.40	1,095	86.0	18.00
14	B	1.50	1.70	0.85	3.50	0.61	0.40	1,130	87.5	20.50
14	C	1.50	1.70	0.85	3.50	0.65	0.40	1,200	93.5	20.50
14	D	1.50	1.70	0.85	3.50	0.70	0.40	1,290	101.0	20.75
14	E	1.50	1.70	0.85	3.50	0.75	0.40	1,380	108.6	20.75
16	B	1.75	1.80	0.90	4.00	0.65	0.50	1,380	106.2	31.00
16	C	1.75	1.80	0.90	4.00	0.70	0.50	1,485	114.8	31.00
16	D	1.75	1.80	0.90	4.00	0.75	0.50	1,590	123.3	31.50
16	E	1.75	1.80	0.90	4.00	0.81	0.50	1,715	133.7	31.50

Table of Weights in Pounds of Special Castings, Metropolitan Water Board:

Diam., Ins.	4	6	8	10	12	14	16
¼ Bends	80	125	185	265	340	455	680
⅛ Bends	60	100	135	185	240	345	440
1-16 Bends	55	85	120	165	210	345	440
Sleeves	45	67	100	120	170	220	275
Caps	25	40	60	80	105	140	240
Offsets	90	175	255	365	490	635	...

Three-Way Branches.—Two Bells.

Size, Ins.	4x4	6x4	6x6	8x4	8x6	8x8	10x4	10x6	10x8	10x10
Weight	120	160	190	215	245	270	285	315	345	380
Size, Ins.	12x4	12x6	12x8	12x10	12x12	14x4	14x6	14x8	14x10	14x12
Weight	355	390	420	460	525	435	475	510	550	615
Size, Ins.	14x14	16x4	16x6	16x8	16x10	16x12	16x14	16x16		
Weight	665	545	580	620	665	730	780	870		

In these two-bell single branches the offset and one end of the

direct main have bells and the other end has a spigot. Such specials often save cutting pipe, but as more or less cutting must occur under any conditions the three-bell branches enable odd pieces to be used, and, in fact, are preferred by most superintendents for general use. Except in the 4, 6 and 8-inch sizes they weigh from 2½ to 4½ per cent. less than the two-bell branches; the small sizes weigh about 3 per cent. more than the all-bell castings.

Four-Way Branches.—Three Bells.

Size, Ins.....	4x4	6x4	6x6	8x4	8x6	8x8	10x4	10x6	10x8
Weight	155	200	245	250	300	345	320	370	415
Size, Ins...	10x10	12x4	12x6	12x8	12x10	12x12	14x4	14x6	14x8
Weight	480	395	440	490	550	665	475	525	575
Size, Ins..	14x10	14x12	14x14	16x4	16x6	16x8	16x10	16x12	16x14	16x16
Weight ...	640	755	835	580	630	685	750	865	945	1100

There are the same differences between the all-bell and three-bell double branches as were mentioned in the case of the single branches.

Weight of Reducers.

Size, Ins.....6 to 4	8 to 6	10 to 6	10 to 8	12 to 6	12 to 8	12 to 10
Weight 75	115	165	185	205	225	250
Size, Ins.....14 to 10				16 to 10		16 to 12
Weight 260				300		330

It is of course out of the question for a foundryman to produce castings of just these weights and some allowance should be made for deviations from the standard weights prescribed. It is a very poor policy to be unnecessarily rigid in the pipe for small water-works. A variation of 5 per cent. under standard weight may well be allowed and any reasonable excess of weight should be permitted, with the understanding that no charge should be made against the city for weights exceeding 4 per cent. of the standard. The weight of each pipe should be determined independently and marked on it in plain letters.

In estimating the amount of pipe needed for any work it may be necessary to take into calculation the slope of the ground; for example, on a hill rising at the rate of one on seven, the horizontal distance determined by surveys should be increased about one per cent. in order to obtain the true length of the pipe line. The cost of pipe varies widely, according to trade conditions; during the civil war it was so high that cement-lined sheet-iron pipe was much cheaper and obtained a strong hold on the favor of water-works designers and officials in the East.

The cost of laying the pipe varies widely, as is but natural in

view of the range of conditions under which the work is done. The figures in the accompanying table for work in Boston afford a rough guide for estimates for the best grade of work under careful supervision. These figures are for lead at 4 cents a pound and labor at $2 a day. Gates and hydrants for pipes 4 to 12 inches in diameter, inclusive, cost about 15 cents per foot of main extra, under the conditions of that city.

Cost of Pipe Laying in Boston.

Pipe.	Lead.	Cartage.	Tools. Gaskets.	Miscellaneous.	Labor.
4 Ins.....	3 cts.	2 cts.	2 cts.	5 cts.	30 cts.
6 "	4 "	3 "	3 "	6 "	35 "
8 "	5 "	4 "	3 "	7 "	40 "
10 "	7 "	5 "	4 "	8 "	45 "
12 "	8 "	6 "	5 "	10 "	50 "
16 "	11 "	7 "	7 "	15 "	60 "

These figures are higher than those given for the same city in Mr. Billings' book, but represent the present condition more accurately.

A uniform rule should be adopted for locating the stop valves on the street mains so they can be found quickly. It will probably prove most satisfactory to place them a few feet back from the building line and thus just off the street intersection, as the roadway at the intersection is worn more rapidly than elsewhere and the tops of the cast-iron boxes reaching up to the surface of the street to furnish access to the valves, project in an unsightly and inconvenient manner. Enough valves should be used so that the breakage of a pipe will cause inconvenience to the smallest number of people possible; it may even be advisable to lay otherwise unnecessary short pipes to connect parallel mains in such a fashion that water can be fed from one to the other in case of emergency. Valves ought to be put on the hydrant branches for the same reason; a 4-inch branch which has been broken during a fire will waste a great quantity of water. The method of conducting such work and of putting in service pipes to buildings is so fully explained in Mr. Billings' book that nothing further needs to be said. The principles which should govern the design of the system of street mains are given in Chapter XIX. There should be plenty of blow-offs at low places so that the pipes can be emptied or flushed out in sections, and if any hills have to be surmounted air valves should be placed at the summits. Air is bound to collect at these places, and there are a number of

different devices on the market for allowing it to escape automatically; if they are not employed trouble generally ensues. It is also desirable to locate gates on the summits whenever practicable. In case of an accident to any one of them water can be drawn off through the blow-offs at the bottom of each of the inclines leading to the summit, and the valve will thus be freed from water pressure for easy repairing. If it was located at a low point this would be very difficult to accomplish.

It is frequently necessary to divide the distribution pipes in the streets into two or more sections, in each of which the elevation varies within limits of 30 to 50 pounds. Each of these sections is commonly called a service district. The reason for this practice is to be found in the necessity of keeping the pressure on plumbing within the limits of its strength; with street pressures over 75 pounds ordinary plumbing is liable to leak and wear out rapidly.

In some cities it is possible to supply the different services from independent sources, but when this is impracticable it is often necessary to let the water from the high-service district pass to the others through pressure reducing valves. These are so arranged that the pressure on the lower side of the valve can never rise about the amount fixed when the apparatus is adjusted. If the draft in the lower district is light the valve allows but a small amount to flow through, but when a fire breaks out and the hose streams are in use the valve opens wide automatically and allows a large volume of water to flow so long as the draft keeps the pressure down..

SUBMERGED PIPE.

Although the chapter on intakes has given some information on methods of laying submerged pipe the subject deserves further consideration. In the first place a distinction should be drawn between pipes having flexible joints and those without. The purpose of these joints is to enable the pipes to follow the profile of the trench by bending without fracture or leakage. The first joint of this nature of which the writer has any record was designed by James Watt for the Glasgow Water-Works Company and used in 1810 in a 1,000-foot line of 15-inch pipe across the Clyde. The bell of this joint was a hemisphere. The spigot fitted closely within it and was cast with the pipe. It was considerably more than a hemisphere, so as to allow bending, and was

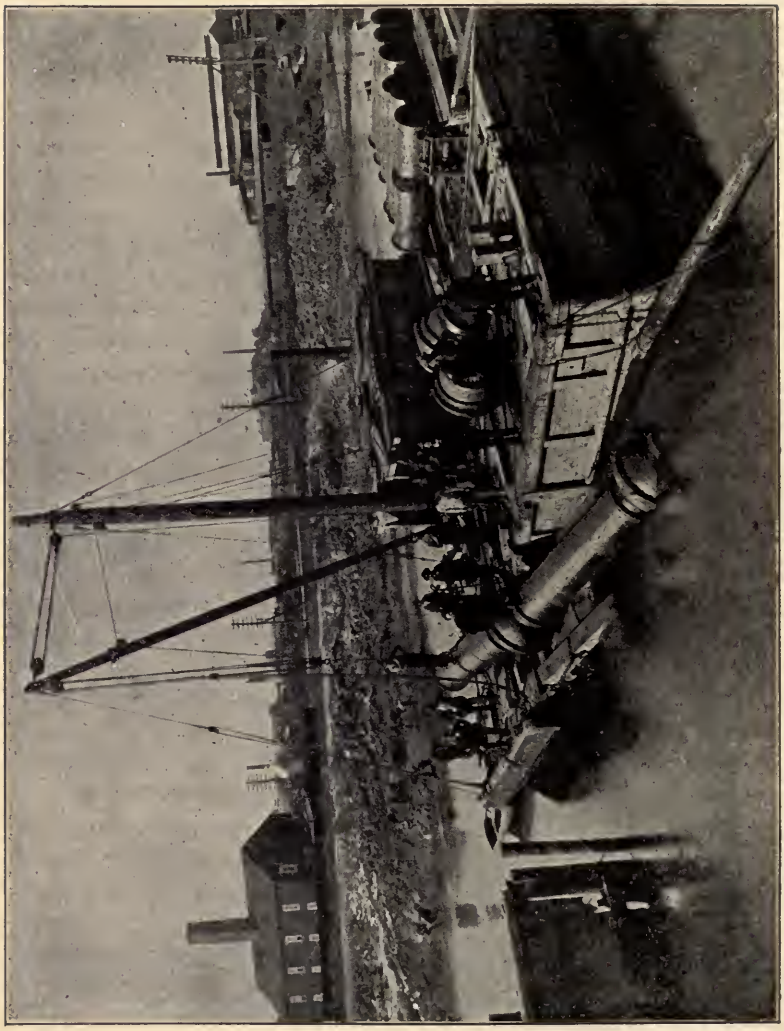


FIGURE 41.—SUBAQUEOUS PIPE LAYING, CAMDEN, N. J.

held within the bell by a lead ring pressed firmly into place by a ring of angle iron bolted to the bell. The joint generally used in this country was designed by John F. Ward, M. Am. Soc. C. E. Most pipe manufacturers illustrate it or some modification in their catalogues and it is shown in Trautwine's Handbook, so it is unnecessary to do so here. All flexible joints are liable to leak somewhat under light pressures, unless they are laid in their final position under a strong tensile strain—which will calk them thoroughly. If the work is to be done in a strong current over a rocky bed, it will be of advantage to use plenty of these flexible joints, 9-foot instead of 12-foot lengths of pipe, and reinforce the bell of each joint with a wrought iron band, about 3 inches wide and 1 inch thick, shrunk firmly in place around the outside rim. It is also desirable to round off the inner edge of the bell slightly so that the lead forming the joint will not be cut when the joint is bent.

The usual manner of laying pipes with such joints was introduced many years ago by Mr. Ward. Figure 41 shows the method as adopted at Camden, N. J. A scow is provided which has a launching way of timbers hung over the side near its bow, the free end trailing on the bottom. A few pipes are calked together on this launching way and the scow is then pulled ahead far enough to allow a new lot to be calked in the end of the first. It is generally unnecessary to make each joint of the flexible type. Sometimes no launching way whatever is used, the end of the pipe last jointed being held by tackle while another section is jointed to its end. This is similarly supported while the scow is pulled ahead a pipe length ready for a repetition of the operation.

A rather unusual method of laying such a main is described in Volume xxxiii. of the "Transactions" of the American Society of Civil Engineers, by Mr. L. L. Tribus, who adopted it in laying a line of 12-inch pipe on the ice in Morris Lake, New Jersey. The ice was 10 to 14 inches thick. A Ward joint was used every 48 feet, there being two lengths of pipe with ordinary bell joints, then a pipe with a Ward spigot, then one with a Ward hub, each length being 12 feet long. These were laid on the ice, blocks being put underneath. It was then lowered by tackle in water from 6 to 12 feet deep, about 50 feet at a time. After it was laid, it could be seen gradually settling in the mud, owing to the clearness of the water.

Sometimes submerged pipe lines are pulled across a waterway from one shore to the other; an instance of this method is described by Thomas H. McCann in the volume of "Transactions" just mentioned. In this case it was necessary to lay a 20-inch force main across two tidal rivers, each about 400 feet wide with a firm mud bottom and slopes of about 12 in 100, the range of tide being about 5 feet. No dredging was done. Ward joints were used, the calking being done on one shore and the pipe then hauled across the river as each joint was completed, a 40-horse-power engine and heavy chains being used for the purpose. As each joint entered the water, two ordinary oil barrels were lashed firmly to the pipe to buoy it. When the end of the main had reached far enough up the opposite shore to allow for the extension due to settlement, the barrels were gradually cut loose and the pipe settled slowly to the bottom. After the water was turned on, a diver was sent down to examine the joints but he found no leaks, and none have since been detected, although an examination has been made every summer by a diver. The main is under a pressure of about 90 pounds. The maximum depth of water in both rivers is 21 feet.

A number of submerged pipes have been laid by Thacher & Shirley, of Toledo, in which a draw joint is employed. In their work part of the pipe is of the usual type and part has a spigot end which is turned to a cone. This cone is inserted in a bell and a lead joint carefully poured and calked about it. The spigot is then withdrawn, leaving the lead in the bell. The pipes are jointed on shore into sections several pipes long, with a turned spigot at one end and a lead filled bell in the other. The ends are closed by bulkheads, and the sections floated with the aid of barrels to the place where they are to be laid. There they are sunk by taking off the bulkheads and casks and lowering them by derricks on scows. When a section is close to the bottom, its spigot end is carefully guided by a diver into the bell of the section already in place. When it is in position, it is drawn into the bell by means of bolts connecting the two ends, and the diver completes the work by calking the joint. Pipes 6 feet in diameter have been laid in this way.

In one instance where a 16-inch submerged pipe had to be laid in about 20 feet of water, the engineer in charge, Mr. E. C. Cooke, employed a light pile trestle on which the pipe was calked in a

profile corresponding to the undulating surface on which it was to rest. The joints were further strengthened by wooden frames clamped tightly about them, so as to secure them from bending and tension. When the main was calked and tied in this manner, it was lowered by slings at each trestle bent.

Pipe laying from a trestle by some modification of this plan is probably the usual method for submerged work across small streams. Owing to the slight flexibility of well-calked joints of the regular bell and spigot type, any slight variation in the rate of lowering at different points of the main is not a serious matter, although, of course, great care should be taken to make the motion uniform at every point. If the bed of the stream is sandy, the current will sometimes cut out the sand below the pipe when it draws near the bottom, and if this is not done a jet of water directed on the bottom will sometimes enable a trench to be quickly excavated, as in the case of the Escanaba intake pipe described in the chapter on intakes. Where the excavation is in soft material and the water is shallow, scoops on the end of long poles have proved the most successful tools for digging the trench in several cases.

The use of rafts of oil barrels is not unusual where submerged pipe has to be laid. After the trench was dug in one such case, two lines of barrels were held by frames so as to form a raft from shore to shore. Along the center of this raft was an open space spanned by transverse wedge timbers on which the pipes were calked together in the usual manner; the main was 16 inches in diameter, 300 feet long and terminated at each end in a bend rising at an angle of about 45 degrees. When it was entirely completed timbers supported by blocks were placed across the pipe at frequent intervals, and the pipe hung by ropes from these timbers. It was first raised a little to allow the wedges to be removed, and was then sunk slowly into place by slacking off the rope slings.

A leak in a submerged water main at Port Huron was repaired in a novel manner by Mr. Hugh F. Doran. The pipe, which was 16 inches in diameter, had been split for about 3 feet by a dredge. A half cylinder of wood, conforming to the outside diameter of the pipe, was made of 1½x2-inch strips or staves, fastened together at each end by a flexible band. On the inside of this wooden cylinder was tacked a sheet of ¼-inch rubber packing, and a sheet of light steel boiler plate was placed on the outside in a similar manner. This patch was then placed over the break by the diver

and securely fastened to the pipe with 4x1-inch wrought-iron bands, made to the circle of the patch and drawn together by 1-inch bolts, the space between the bands being 7 inches. By this means the pipe was made perfectly tight and has repeatedly withstood fire-pressure.

It sometimes happens that a pipe must be carried over a stream or gorge, rather than under it, and it is thus directly exposed to the weather. If the pipe is less than 500 feet in length, practical experience in New England shows that the lead joints will take up all the variations in length due to temperature changes. Usually the pipe can be hung from a bridge, but if it cannot and the span is short, it may be trussed by tie rods and struts below it so as to form the upper chord of a queen post truss of sufficient strength to support its own weight and that of the water it contains. This has been done at several places, but the only case of which the writer has definite records was at Lynchburg, Va.

If it is certain there will always be a flow of water in the pipe there is no reason for protecting it by a non-conducting covering, although this is usually done. The usual plan is to cover the pipe with hair felt, then leave an air space all about it and finally put a sort of double housing over it with the space between the outer and inner planking filled with sawdust. The housing must be water-tight to be of much use.

CLAY PIPES AND OPEN CHANNELS.

It is sometimes possible to conduct water for a considerable distance along the hydraulic grade line—and to substitute a vitrified clay pipe line, timber flume or open channel for the more expensive iron pipe. The velocity of flow in a vitrified clay pipe line may be estimated by means of the following formula, deduced by Mr. Rudolph Hering from the Kutter formula.

$$v = A r \sqrt{s} \div (B + \sqrt{r}).$$

In this formula s is the slope or grade, v is the velocity in feet per second, and r is the hydraulic radius, the quotient of the cross-section of the stream of water in square feet divided by the wetted perimeter of the channel in feet. For vitrified pipe A may be taken as 188 and B as 0.64. For a carefully made flume of planed timber A may be taken as 200 and B as 0.55. The discharge in cubic feet per second is found by multiplying the velocity by the cross-section of the stream. The flow through an open paved

channel of small size depends on so many conditions difficult to specify exactly that it would be idle to discuss the subject here.

Vitrified clay pipe for water conduits were first brought prominently before engineers in 1888 by Mr. S. E. Babcock in a paper before the American Water-works Association describing such pipe lines in the water-works systems of Amsterdam, Little Falls and Johnstown, N. Y. In no place are they more than a couple of feet below the hydraulic grade, and they form part of a system in which open channels and cast-iron pipe are also employed. In the plant at Little Falls the pipe was required to have a thickness of one-twelfth of the diameter and be fitted with hubs 3 inches deep and large enough in diameter to allow for a ⅝-inch cement joint all around the circumference of the spigot. Five per cent. variation in the dimensions was allowed during inspection. The pipes were well glazed all over and any which had a fire crack considered injurious by the engineer were rejected. Pimples and blisters on the interior surface liable to check the flow of water, were other grounds of rejection. Pipe with slight cracks or breaks in the socket were accepted, for such defects cause no trouble in a conduit not subject to internal pressure of any appreciable amount.

The method of laying is best indicated by quoting from the specifications:

"The joints of the vitrified pipes shall be made of Portland cement mortar in combination with gaskets of clean, sound hemp yarn or jute, braided or twisted, and tightly driven, as follows:

"Each length or strand of the jute shall be of a diameter to loosely fill the width of joint, and shall be thoroughly soaked in a Portland cement mortar, made of thick paste of clean cement and water, and shall be of a length to go once around the circumference of the pipe and lap over two or three inches. This shall be driven home with calking tools, and shall be succeeded by a sufficient number of strands to fill the joint room to within one-half an inch of the outside of bell, breaking joints with the laps. All driven home and thoroughly joined together. The joint shall then be finished by filling the remaining one-half inch of joint room with a clear Portland cement mortar, the joint room when finished being completely filled all around the pipe to the outside lines of the bells.

"The contractor will furnish the pipe layer with a bag, stuffed with shavings or hay, of a size sufficient to fit the pipe rather

tightly, with a rope about ten yards in length fastened at one end to the mouth of the bag. The bag must be placed in the first pipe, the rope passing through each pipe as it is laid down. After the joints are made, the bag is then to be drawn forward at such times before the cement has set as to smooth off and produce a true surface at each cement joint and a continuous thin coating of cement on the lower half of the pipe."

These specifications call for a good class of sewer pipe laying, and there should be no difficulty in having them carried out, or others of an equivalent nature. The cost of the Little Falls conduit, consisting of 10,000 feet of 20-inch vitrified pipe on a grade of 8 feet per mile, 18,500 feet of 18-inch on a grade of 13 feet per mile, 900 feet of 15-inch on a grade of 79 feet and 1,000 feet of 12-inch on a 105-foot grade, was $45,544, or approximately $1.50 a foot, while the portions of the same conduit which were laid with cast-iron pipe cost nearly $2.60 per foot. In the ten years the pipe has been in service it has been free from breaks.

Such a conduit must have no valve or gate except at its upper end, for if the flow of water in it should be checked it will be subjected to a hydrostatic head which may cause serious trouble. If there is no gate anywhere along its line, such danger is reduced to a minimum. The pipe must discharge into a well or reservoir of some kind, for the same reason.

Open channels for conveying water for domestic purposes have the serious fault of allowing grass, leaves and other impurities to enter the supply, and generally permit a considerable proportion of the water to become lost by percolation into the earth. On the other hand, their small cost makes their use advisable where the water is ample in quantity and not exposed to contamination during its flow through the channel. It has been found in some portions of the West that an open channel lined with cement to prevent percolation is the cheapest satisfactory method of distributing water over sandy plains for irrigating purposes; and in a number of water-works, open paved channels or beds of natural brooks are employed. Flumes of plank calked with oakum are used extensively in the West, but the cost of constructing them for small quantities of water will probably be equal or greater than the expense of vitrified clay or spiral riveted pipe of the same capacity.

If an open channel can be introduced to advantage, care should be taken to avoid such dimensions that the velocity will wear away

the sides or bottom, if they are of earth. Where the slope is too great for an open channel to be constructed in one unbroken line, it may be fitted with a series of falls between which a satisfactory grade can be secured. Just before each fall is reached the channel should be narrowed somewhat to allow for the greater velocity acquired by the water before it drops. An apron of some sort, or a water cushion, should be provided to resist the erosion of the water where it strikes, provided the quantity is large or the fall is great. Open channels are used in connection with vitrified pipe lines on both the Amsterdam and Little Falls conduits, and in the case of the latter one of these channels, which is quite steep, has a series of low dams or riffles at frequent intervals, which were introduced to secure a thorough aeration of the water.

Where an open channel carries water in which considerable sand may be held in suspension, it is well to pass it through a sand catcher. This is merely a chamber or basin large enough to check the velocity of the water considerably. The result of such a diminution of velocity is the settlement of the sand and coarse silt to the bottom of the chamber, which should have a fairly large blow-off pipe fitted with some sort of a sluice gate. When enough sand has been intercepted to warrant clearing the basin, the gate is opened and the sand flushed away through the pipe to some place where it will do no harm. An ordinary gate valve is not adapted for such use because it is liable to have the valve cut by the sand when it rushes out of the chamber. The blow-off pipe should be of fairly large diameter so as to empty the chamber quickly, but not so large that the velocity of the water will be inadequate to clean out the sand.

In case it is desired to calculate the flow through an open channel, much assistance can be secured from No. 84 of Van Nostrand's Science Series, "The Flow of Water in Open Channels, Pipes, Etc." by the late P. J. Flynn.

CHAPTER XVIII.—SERVICE RESERVOIRS AND STAND-PIPES.

The bona-fide direct-pressure system of water-works has now passed into history, for no one acquainted with even the elements of water-works management thinks of building a plant without a small reservoir, stand-pipe or elevated tank to supply the sudden large demands for water for fires, while the pumping machinery is being speeded up to the increased duty. It is now settled practice that every works depending upon pumping should have some sort of a service reservoir or stand-pipe. It furnishes a supply of water during the night, so that the pumps may be shut down and the fires banked, thereby saving coal and attendance, and it equalizes the head against which the pumps work and makes their operation more economical in consequence. Even with gravity supplies service reservoirs are often advisable, as their construction may cost less than the difference in expense between a long conduit large enough to supply the maximum demand for fire and domestic purposes and a smaller conduit ample, with a storage reservoir, to furnish the same service.

Reservoirs can, of course, only be employed where the topography is such that an elevated site is available for their construction; if the country is flat a stand-pipe or water tower must be used. If a site can be found, the first question to be settled is the storage capacity to be given the basin; if the cost of construction will be comparatively small, storage for several days should be provided. In this connection attention should be paid to the fact that such a reservoir has a decided influence on the size of the pumping plant and force main. These need only be designed to furnish the required volume of water when working steadily during the daytime. Uniform operation under these conditions means that the size of the plant and main can be kept down, the cost of operation reduced, and but one force of attendants need be employed. The pipe leading from the reservoir must be sufficiently large to supply

the maximum domestic and fire draft, and may therefore be of larger diameter than the force main.

If the water comes from wells it should be protected from the sunlight, which makes some sort of covering necessary at the standpipe reservoir. The heavy masonry roofs of foreign plants are too costly for this country, and the writer believes that the least expensive device for small work will probably be arched vaults of brickwork or the use of light steel I beams with terra-cotta filling such as is used for floors in high buildings. The latter plan has never

FIGURE 42.—ELLIPTIC GROINED ARCHES, ASHLAND.

been tried, to his knowledge, but some rough estimates indicate that it is worthy of consideration. These roofs can be covered with earth and will keep the water from freezing except in very severe weather. If the roof is to be above ground, wood or iron framing and shingling are doubtless as good as more expensive concrete arches, for ice will probably form during cold weather no matter what type of construction is followed.

The groined brick arches used at Ashland, Wis., by Mr. William Wheeler, and shown in Figure 42 are among the most interesting masonry roofs built in this country. The brick piers are 15¾ feet apart in the clear and the elliptical arches have 3½ feet rise. In a description of this work in the "Journal" of the New England

FIGURE 43.—SUPPORTS FOR ARCHES, NEWTON RESERVOIR.

Water-Works Association, Mr. Wheeler states that the arch rings are about 5 inches thick and consist of two courses of bricks laid flatwise in Portland cement mortar. The spandrels of the arches and the spaces over the piers and adjacent walls are filled and covered with a backing of Portland cement concrete up to a general level of 4 feet above the spring of the arches, but sloping down to a height of only 2 feet above the springing line at the rear of the outside walls. The bricks in the arches are laid in uniform horizontal courses, and cut or mitred to fit the angles of their intersections, except in a few courses next the springing lines and also at and near the crowns, where the corresponding courses of the intersecting arches are neatly bonded with or into each other without cutting. There are openings at the alternate intersections of the arches to afford access to the space below. Upon the concrete backing which overlies the covering arches 2 feet of earth has been placed and seeded.

The construction of such vaulting may give so much trouble to inexperienced contractors that a simpler system may prove desirable in some cases. In Figure 43 is shown the system of brick piers and lintels built at Newton, Mass., by the late Albert F. Noyes to support a roof of parallel brick barrel arches. It is probable that most masons would prefer to build such a roof and might put in a lower tender for it, although it requires more masonry than the groined arch which is really little if any more difficult to construct if a competent engineer is on hand to start the work properly.

Service reservoirs must be lined on the bottom and slopes to make them water-tight. Formerly nothing but puddle, protected on the slopes by dry stone paving, was used for this purpose. Puddle of good material properly laid under the direction of an experienced engineer will make a tight basin. Unfortunately, however, if the slopes are steep or the puddle contains much clay, the slope lining may slide to the bottom when the basin is emptied after a long period of service. Moreover, really good puddle material is hard to find in many localities, and, even when found, it is not always well made.

It was but natural, therefore, for engineers to line many of these basins with concrete. Frequently puddle is first laid and then the concrete placed over it. Well made concrete is an excellent material for this purpose; but unfortunately it is not always well made. Even when of good quality it must be placed in such a manner

that the puddle or earth on which it rests cannot be washed from below it by percolation down the slopes under the lining or by the pressure of the stored water. Of late years engineers have sought additional security by covering the concrete with asphalt, and, in some cases, have abandoned the concrete entirely and used brick and asphalt in its place.

CONCRETE-LINED RESERVOIRS.

Concrete of a high grade is not often laid in large continuous sheets, and some engineers hold that it cannot be done without cracks forming. As a matter of fact, continuous concrete reservoir linings have been constructed without such cracks appearing, and, owing to the slight range of temperature of such a lining and consequent freedom from expansion and contraction, the writer believes they should be built if it seems probable good results can be

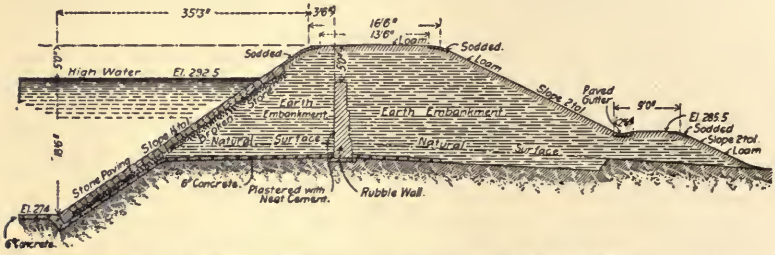


FIGURE 44.—HAVERHILL RESERVOIR EMBANKMENT.

obtained. If the work is done by contract, the cement should be furnished by the city, not only to insure the use of good material but also to keep the contractor from skimping the quantity. One of the most notable instances of the successful construction of a continuous lining of this sort is afforded by the Palatino reservoir of the Havana water-works, of which Mr. E. Sherman Gould was the engineer. Here one foot of concrete was laid in two 6-inch courses over an area of about 2.8 acres. The best quality of Portland cement was used, and the concrete was mixed in the proportion of one part of cement, three parts of sharp calcareous sand and five of broken limestone. The work was kept scrupulously clean, and the finished concrete was sprinkled with water by hose lines and sprinkling cans for one to two weeks after it was laid. The result was completely successful.

Such expensive construction is not usually followed in the United States, even in large works. At the service reservoir of

the Syracuse water-works, for example, the concrete lining was but 9 inches thick and made of one part of hydraulic cement, two parts of sand and three of stone.

The high-service reservoir built from the plans of Mr. Freeman C. Coffin in 1898 to store 9,000,000 gallons of water for supplying a part of Haverhill, Mass., may be taken as an illustration of good practice in the construction of such basins. The basin has a maximum depth of water of about 19 feet and is partly in excavation. It is generally advisable to utilize the excavated material if of suitable nature in the banks, as was done in this case, as such

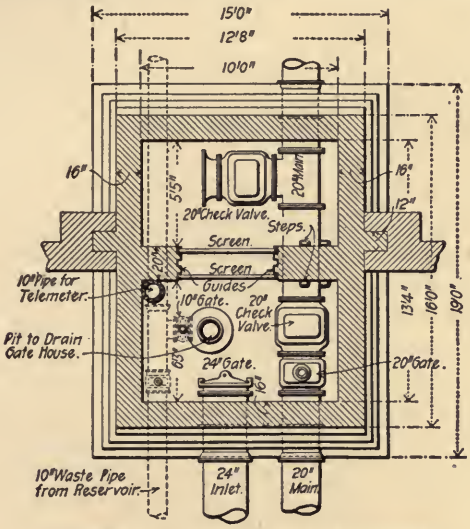


FIGURE 44A.—HAVERHILL GATE-HOUSE.

a course tends to reduce the cost of the work. A cross-section of the embankment is shown in Figure 44, and the plan of the gate house in Figure 44a. A brief explanation should be added concerning the gate-house piping. The 20-inch supply pipe is carried on brick piers across the reservoir to the opposite side from the gate-house. The 24-inch inlet pipe is taken from the foot of the slope just in front of the gate-house. The 20-inch pipe acts as a supply to the pipe system when the pumps are not working. It has a gate and check valve which prevents the water from running back into the main without first passing through the 24-inch pipe and gate-chamber. When the pumps are not working the

water runs through the 24-inch pipe into the gate-chamber, where it passes through screens and then through a check valve, placed on an offset, into the 20-inch pipe. There is a vertical 10-inch pipe in the gate-house for a telemeter. There is also a 10-inch waste pipe with a branch to drain the gate-house, which passes beneath the latter in a bed of concrete. The 20-inch main is embedded in concrete between the gate-house and the outer edge of the embankment. The concrete around each of these pipes is rectangular in cross-section, 6 inches thick above and below them, and 10 inches thick on the sides, with two cut-off walls.

The following extracts from the specifications make further explanations unnecessary:

"Earth Work.—All the ground covered by the reservoir and its embankments shall be cleared of trees, stumps, stones, roots, turf and other vegetable matter, which shall be separately stored in spoil banks at such points as the engineer may direct. The soil is to be removed from the ground to be covered by embankments, and stored in spoil banks at convenient points where directed for subsequent use on the surface of embankments. Any further amount of spoil that may be required shall also be stored in spoil banks for future use.

"All embankments or fills shall start from a well prepared base, fitted for incorporation with the filling, and shall be formed of earth free from roots, muck, stones measuring more than 3 inches in any diameter, perishable earth or other unfit material. The small stones allowed to go into the embankment shall not form more than one per cent. of the material, and shall be so disposed as not to be liable to come in contact with each other. The surface soil suitable, in the opinion of the engineer, for the purpose is to be thoroughly mixed as he may direct with other material and used in forming the banks.

"If in the opinion of the engineer other earths found in the excavation require to be mixed to form a suitable embankment, they shall be mixed in such manner and so disposed as he may require. The embankment shall be carried up in layers, slightly concave in cross-section but level longitudinally, not exceeding 4 inches in thickness before rolling, every layer to be carefully rolled with a heavy grooved roller, and watered more or less when and as required. No lumps will be allowed to go in, and all possible care taken to make the embankments impervious to water. The earth is

to be well and solidly rammed with heavy rammers at such points as cannot be reached with the roller. The embankments are to be overfilled as required, in no case more than 12 inches on the interior slopes, which shall afterwards be dressed off and will be reckoned as excavation.

"Soiling the Surfaces and Slopes.—The soil that is stored in spoil banks as above stated shall be placed upon the top and slopes of the reservoir in a layer of such thickness as the engineer directs. It shall be rammed if required by the engineer and rolled with a heavy hand roller and trimmed true to grade. The slopes shall be sodded as shown on the drawings. The sods to be approximately one foot square, not less than 3 inches thick, and have a good heavy growth of grass and roots. Each sod shall be pinned to the bank with a wooden pin not less than 15 inches long. The sodding shall be sprinkled when necessary in the opinion of the engineer.

"Puddle.—The puddle shall be composed of good pure clay and clean gravel in such proportions as shall be satisfactory to the engineer. If at any time the clay provided by the contractor will, in the opinion of the engineer, bear any addition or admixture of any materials from the excavation, the engineer shall determine what proportion of such material shall be used, and from what part of the excavation it shall be taken, and in what manner it shall be mixed with the clay. There shall be no material from the excavation used except with the approval of the engineer, and he shall have the right at any time to reject any clay or material for puddle that is not satisfactory to him or that will not in his opinion be suitable for the uses for which it is intended, and such material shall be immediately taken away from the work.

"The bottom of the reservoir shall be prepared for the reception of the concrete or puddle in a manner satisfactory to the engineer. If the bottom is in rock or ledge it shall be filled up to the grade of the under side of the puddle or concrete and well tamped or rolled before the puddle or concrete is applied. If there are cracks or fissures in the ledge they shall be filled with concrete or mortar.

"Reservoir Lining.—The reservoir will be lined over the bottom, on the inner slope and in the embankment as shown on the drawings, with a layer of hydraulic cement concrete of such thickness as the engineer directs, not less than 4 inches. This concrete shall be put on in one layer, carefully leveled up and lightly ram-

med until the mortar flushes to the surface; then surfaced with cement mortar made as specified herein. The surface shall be immediately finished off with trowels, using only enough mortar to smooth up and make a fine, close surface. In finishing this surface, planks shall be laid upon the concrete for the masons to walk and work upon. As soon as any portion is finished it shall be at once guarded from any disturbance. The bottom shall be the last part of the reservoir to be finished, and shall not be done until special orders are received from the engineer. The concrete shall be protected while setting from the sun and rain by canvas or otherwise in a manner satisfactory to the engineer. [It might be well in most cases to require concrete to be sprinkled also, when the engineer so directs, as hot dry weather may injure new work even when covered by canvas or boards.]

"Core Wall.—There will be a core wall of rubble masonry in the embankment around the reservoir. This wall will be of rubble masonry as specified. It shall be laid against wooden forms on the inside face, and the stones shall be laid with at least one-fourth of an inch between the forms and their extreme points. All voids between the stones and the forms shall be filled. No stone shall be laid in such a manner as to project entirely through the wall. These forms shall be approximately 3 feet high and 10 or more feet long, and sufficiently strong to retain their shape. There shall be a sufficient number of them to allow the cement to set not less than 14 hours before their removal. Any voids or rough places in the face of this wall shall be pointed up with cement mortar. This wall shall be built at all points upon natural undisturbed earth from which all vegetable matter and loose or soft material have been removed. It shall be carried up at about the same level all around and protected from the sun and rain by canvas or otherwise, in a manner satisfactory to the engineer.

"Plastering.—The face of the core wall shall be plastered with a thin coat of neat cement mortar which shall be rubbed in and thoroughly compacted and smoothed with trowels. If directed by the engineer the face of this plaster coat shall be washed with neat cement grout. The plastering shall be protected from the sun and rain in a manner satisfactory to the engineer.

"Broken Stone.—After the bank is carried up the inner slope will be dressed true to line, and a layer of broken stone or screened

gravel, averaging about 6 inches in thickness, will be applied. The stone used for this purpose must be sound, and shall be of such size that they will all pass through a screen of 3-inch mesh and none of them will pass through a screen of 1-inch mesh. The various sizes must be well mixed in due proportions, and when laid in place shall be well compacted by ramming. All broken stone so used must be freed from fine material by screening.

"Slope Paving.—The inner slope is to be paved with sound, selected stone of good shape to make tight, firm work, and laid dry. The stones are not to be less than 10 inches in any dimension, and not less than one cubic foot in solid contents. The paving shall be taken from ledge or boulders large enough to make at least two pieces. The split pieces shall form the face of the slope whenever possible. Stones with flat faces may be laid without splitting, when so directed by the engineer. They shall be laid as closely as practicable. Each stone shall have a firm bearing in the broken stone backing, and shall be thoroughly pinned, and every precaution shall be taken to make each stone secure in its place. The lower course of the footing of the slope is to have stone of the full depth of the surface paving and the broken stone backing, and it is to be laid in such manner that its top surface shall be parallel to the slope of the embankment. This lower course shall be laid in cement mortar.

"Mortar.—The sand shall be clean, sharp and free from loam. The proportions of cement to sand shall be those designated by the engineer for different parts of the work. They shall be carefully mixed in the following manner: About one-half of the sand for a batch shall be spread evenly upon a tight and smooth platform, with low sides to prevent the washing away of the cement. The cement shall be spread evenly upon this layer, and the balance of the sand spread on top. The whole shall be turned with shovels and thoroughly mixed dry. Water shall be applied by moderate sprinkling from a sprinkler nozzle. Care shall be taken to avoid an excess of water at any time, and the total amount applied shall only be sufficient to make the mortar of the proper consistency for the work in hand. The size of the batch to be wet at once shall be as directed by the engineer. The mortar shall be freshly mixed and any mortar that has been standing long enough to begin to set shall not be used.

"Concrete.—The concrete shall be formed from broken stone

or sound angular stone screened from gravel. The sand screened from the stone will be used when considered suitable by the engineer. In screening the material, three screens shall be used; namely, one with 2-inch mesh clearing opening, one with three-quarter-inch mesh, and one sand screen, the size of mesh to be satisfactory to the engineer. All material not passing the 2-inch screen will be rejected. All passing the 2-inch and retained on the three-quarter-inch screen will be used if suitable in other respects. All material passing the three-quarter-inch screen and retained on the sand screen to be used when and where ordered by the engineer. The material passing the sand screen to be used for sand if considered suitable by the engineer.

"Mixing Concrete.—The gravel, which must be clean and free from dust and dirt, shall be spread upon a smooth, tight platform in a movable frame. This frame shall be of gauged dimensions for holding the proper amount of stone for a batch according to the direction of the engineer. The gravel shall be leveled off with the top of the frame and thoroughly wetted. The cement and sand, mixed dry upon an adjacent platform as specified for mortar, shall be uniformly spread upon the top of the stone. The frame shall be lifted off and the material carefully and thoroughly mixed by being turned over with shovels, the men working systematically under the direction of the engineer. If additional water is required, it shall be moderately sprinkled on the material from a sprinkler nozzle, care being taken not to wash out the cement or to put on at any time an excess of water and to leave the concrete too dry rather than too wet. If machine mixing is adopted, the machinery and the method shall be satisfactory to the engineer.

"Placing the Concrete.—All concrete shall be placed in horizontal layers about 6 inches in thickness unless otherwise directed by the engineer. It must be deposited in such a manner that there will be no separation of coarse from fine material. It shall be lightly rammed until the mortar flushes to the surface. It shall be deposited quickly after it is mixed, and as far as possible the placing of concrete shall be continuous. In joining new work to that which is set or practically set, such precautions of cleaning, wetting and bonding shall be observed as the engineer directs.

"Rubble Masonry.—All stone masonry is to be laid in mor-

tar such as above described, and shall be well bonded with sound, angular stones, laid on their natural beds, and so as to break joints sufficiently for strong work. All parts of the walls are to be carried up at a uniform rate, so that the entire surface shall be at about the same level as the work progresses. The stones are to be properly moistened before laying in mortar. Rubble masonry shall be so laid as to be absolutely without voids. Each stone shall be completely surrounded with mortar. All joints shall be as small as the nature of the stone will allow."

These specifications have been quoted at such length for the reason that they are somewhat out of the ordinary in placing practically complete control over the concrete work in the hands of the engineer. While this is frequently done in specifications of different form, nevertheless the manner in which it is accomplished by the clauses which have just been quoted, tends to produce far better results than the more usual method of specifying the proportions of the various materials entering into the concrete and requiring the contractor to furnish them all. Where a contractor is paid a lump sum for the construction of so many cubic yards of concrete and he furnishes the cement, he is exposed to the constant temptation to use as little cement as possible. Moreover, unless the basis of payment is on the number of units of the various classes of work in the entire undertaking, there is a natural tendency on the part of the contractor to do as little work as is necessary to comply with the letter of the specifications. Where he is paid by the cubic yard of excavation, the cubic yard of concrete, etc., the more work the engineer calls for the larger will be the profit on the entire contract, provided, of course, the contractor has not attempted to balance his bids in any manner.

A combination of concrete and brick is sometimes used for the lining of reservoirs, an example of such work being shown in Figure 45, which illustrates the construction followed in 1898 in a reservoir built at Lancaster, Ohio, from the plans of Mr. John N. Wolfe. It measures 100x200 feet at the coping level and is nearly 19 feet in depth at the lowest point. The reservoir had to be built on a soft sandstone formation overlaid by clay, and a low masonry wall was constructed in order to secure the desired surface elevation. In this case the bottom and sides of the basin were covered with 8 inches of concrete on which a course of brick was laid flatwise. This course was then covered with 1 inch of

cement and lime to form a cushion for an inside lining of brick laid edgewise in cement.

ASPHALT-LINED RESERVOIRS.

The use of asphalt as a waterproofing material is a revival of the practice in vogue centuries ago. Mr. Rudolph Hering has pointed out that this material was used at Babylon and Ninevah about 4,000 years ago as a mortar and also mixed with broken bricks and stone to form a concrete. It was used in Egypt, particularly in Memphis, for keeping moisture out of walls and basements. A German pamphlet of the early part of the seventeenth century describes the application of the material for various purposes, but it was a Greek physician, Dr. Eyrinis, who first brought

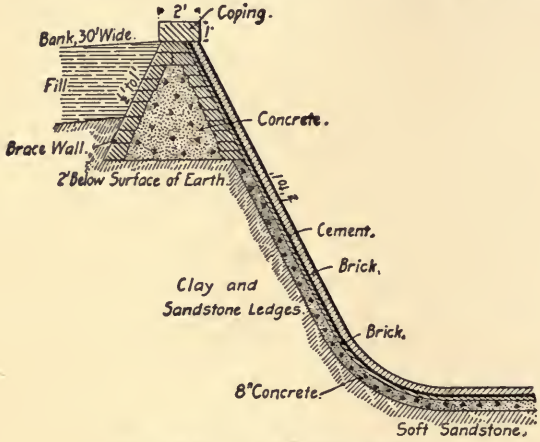


FIGURE 45.—LANCASTER BASIN.

about its modern use for waterproofing. There are records of cisterns 16 to 20 feet in diameter which were laid in asphalt and retained water successfully. Count Buffon, the French naturalist, recommended asphalt as a mortar in building a large basin in the Jardin des Plantes in Paris, and some forty years later he wrote that it had remained perfectly water-tight. The present practice in the use of asphalt in America may be best illustrated, since it varies considerably, by describing a number of recent reservoirs in which it has been used.

The Astoria, Ore., reservoir, built under the direction of Mr. A. L. Adams, M. Am. Soc. C. E., is lined on the bottom with concrete and asphalt. The concrete was made with Portland cement and laid in 20-foot squares 6 inches thick, the joints between the

squares being about one-half inch thick. The joints were run full of asphalt when the first coat of the latter was applied. The asphalt was of the Alcatraz brand of the L and L L L grades. When the concrete was at least two weeks old it was covered with the L or liquid grade and then followed by a layer of the L L L or hard grade. Mr. Adams believes that any advantage possessed by a soft coat over a harder one, as a first application, is more fancied than real. "At the proper temperature, the harder grade runs just as readily, and enters all crevices just as surely when applied and if the masonry be entirely dry and clean, and preferably a little rough, it adheres more tenaciously than the liquid. The only superiority possessed by the latter as a first coat is that if it must be applied on a damp surface, it will adhere where the other will not."

On the slopes the concrete was applied in sheets 10 feet wide extending up and down the slopes. This was covered with a layer of the L L L asphalt, and paved with bricks dipped in buckets of hot asphalt and placed in position with tongs. A final finishing coat of asphalt completed the work. "When this lining had been exposed to the rays of the sun for a considerable time, there was a very noticeable sliding of the brick on the slope, and a consequent closing up of the joints, crowding out the asphalt where they were thick by reason of the asphalt having been used too cold on days that were too windy to permit it being maintained at a proper temperature. As a consequence, openings were produced between the brick lining and the wall at the top of the slope, which were filled with mastic. The footway course of brick was prevented from sliding by being set into the concrete of the bottom lining."

Rock asphalt mastic was used in repairing a leaking reservoir at Coatesville, Pa., a work for which Mr. Alexander Potter was the engineer. In this case an old brick lining was taken out and the clay puddle below it thoroughly drained. Brick was then laid over the bottom and slopes, and painted with a thin coat of refined bitumen and benzine. Over this the asphalt was spread 1 inch thick on the bottom and ¾ inch on the sides. It was placed in two layers and was Neuchatel mastic heated to a temperature of 400 degrees Fahrenheit, poured on the surface from wooden buckets and smoothed with wooden spatulas. The asphalt is carried to the head of the slopes and 18 inches along the level top, where it is joined with a concrete gutter 15 inches wide and 4 inches

deep. This intercepts the water which might seep down the slopes below the lining and eventually injure them. Mr. Potter has stated that the use of a somewhat thinner asphalt lining covered with tiles about 1 inch thick might prove even more satisfactory than the plan described. These tiles should be dipped in refined bitumen and laid before the asphalt cools.

Mr. Gervaise Purcell, Assoc. M. Inst. C. E., has used an asphaltic concrete reservoir lining which he describes as a mixture of 25 per cent. of sand and 75 per cent. of gravel in 100 pounds, added to 10 pounds of California asphalt mastic. The sand and gravel are heated to a temperature 310 degrees and then mixed thoroughly with the asphalt, which has been heated separately to a temperature of 280 to 300 degrees. This concrete is tamped into place, and then smoothed with a hot roller to prepare it for a surface coat, prepared according to the following formula: "To 100 pounds of hard asphalt are added 3 pounds of liquid asphalt capable of standing a 300-degree flash test with specific gravity under 10 degrees Beaume; the whole is heated to 400 degrees Fahr., and the temper maintained by the addition of small quantities of liquid asphalt from time to time as the more volatile portion evaporates; to this is added between 10 and 12 per cent. of powdered carbonate of lime or sulphate of lime, according to the peculiarities of the asphalt." This is poured over the surface and rubbed smooth with a heated iron tool 2 inches in diameter, 8 inches long and mounted in a long handle.

The reservoir of the La Grande, Ore., water-works was built in 1892 from plans prepared by Messrs. Adams & Gemmell. It is an oval basin of 1,000,000 gallons capacity, partly in excavation and partly in embankment, and is lined with a course of brick covered by a $\frac{3}{8}$-inch coat of asphalt. After the slopes were thoroughly rammed, the bricks were laid on edge and their joints filled with clean sand. The asphalt was cooked at the site of the work, its proportions being one part by weight of coal tar to eight or nine parts of California rock asphaltum. Between 1,700 and 1,800 pounds of asphalt were cooked at a time, the process lasting five or six hours, or until the mass became liquid and uniform. It was spread over the bricks by means of shovels and brooms, two layers being necessary to make up the requisite thickness. The total cost of the lining, brick work, asphalt and labor, was about 12.5 cents per square yard.

Another reservoir, of about 420,000 gallons capacity, built by the same engineers at Waitsburg, Wash., was likewise partly in excavation and partly in embankment and dependent for tightness on an asphalt lining. The earth was light and ashy, and was difficult to work into a bank. The lining was put on in two layers, one of paving asphalt 1½ inches thick, and the outer one of pure asphaltum about ⅛-inch thick. The asphalt was mixed in the proportions of eight parts rock asphaltum to two parts Las Conchas asphaltum. These ingredients were cooked from ten to twelve hours, and then mixed with sand in the proportion of one part by weight of asphaltum to five parts of hot sand. The asphalt was laid in vertical strips, the edges of the strips being painted with pure asphaltum before new material was laid against them. On account of a leak in one of the pipes, the earth below a part of the lining was undermined a few days after water was turned into the reservoir. The water pressure broke the lining, but not until the asphalt had been dished 4 inches in an area perhaps 20 inches in diameter. The cost of this lining was the same as that at the La Grande reservoir.

A true asphalt concrete has been recommended by Mr. R. B. Stanton, M. Am. Soc. C. E., for reservoir linings under certain circumstances. The concrete used by him on a reservoir for mining purposes consisted of porphyry broken to 2 inches and less in size, with all the small stone and dust left in and enough fine material added to form a perfect concrete when bound together by the asphalt. The asphalt was formed by four parts of refined California asphaltum and one part of crude petroleum, cooked in a paving kettle until pasty and then poured over the rock, which had meanwhile been heated in another kettle. This concrete was put on in a layer 4 inches thick in strips from 4 to 6 feet in width, the old edges being coated with hot paste before new material was laid against them. When the lining was finished it was painted with hot asphaltum paste mixed in the same proportions as that for the concrete, but boiled a much longer time, until when cold it was hard and brittle, breaking under the hammer like glass, yet tough, elastic and pliable with the least warmth. This painting was done while the paste was very hot and could be ironed down with hot irons. Its thickness should not exceed ⅛ inch. This lining gave excellent satisfaction in difficult service.

Another method of using asphalt recommended by Mr. L. J.

Le Conte, M. Am. Soc. C. E., for steep slopes is the following: The slope of wall of masonry is first coated with asphalt heated only enough to make it liquid, which has great penetrating and adhesive properties, but is lacking in sun-proof qualities. Then a layer of ordinary heavy burlap is stretched tight and pressed into the asphalt paint. The final step is to put on a boiling hot coat of "hard" asphalt, which constitutes the weather surface and is hard, tough and admirably resistant to the hot summer sun. "Wherever this lining has been used, no signs of creeping have developed even on smooth vertical faces. Hard asphalt paint is lacking in adhesive qualities, and consequently cannot be placed directly on the slopes." This lining is recommended for slopes steeper than 1 on 1.5; for flatter slopes Mr. Le Conte advises the use of asphalt mortar or asphalt cement.

The latest example of an asphalt-lined reservoir at the time of the writing of this chapter is afforded by a 1,600,000-gallon basin constructed at Black Hawk, Colo., under the direction of Mr. Walter Pearl. Most of this basin is in excavation, but at some places the slopes have been carried upward by rubble masonry walls backed with earth. The lining is brick dipped in Pacific asphalt paving cement. Most of the brick used on the bottom are what is known as machine made, medium hard burned, sand brick; those used on the sides are terra-cotta brick of the usual building size. The side slopes are four on one. At least one-third of the excavation on the slopes was left in such bad condition by blasting that it had to be lined with rubble masonry laid in Milwaukee cement before the brick could be placed. The bottom of the reservoir was covered with a cushion of sand from $\frac{1}{4}$ to 3 inches thick, varying according to the inequalities of the surface of the excavation. The brick laid on it were dipped for half their width in hot asphalt, and after they were placed on edge on the sand the joints and cracks were filled with asphalt, which material was also swept over the surface with brooms.

Around the edge of the bottom lining a bed was made to furnish a footing for the slope lining. This bed was a mixture of about 20 per cent. of asphaltic cement and 80 per cent. of sand. Before the brick were laid anchor bolts were placed in the walls so that the brick could be held from falling inward after they were laid. The brick for the sides were entirely dipped in asphalt except for the small space where they were held. After they were placed

they received a coating of asphalt. The brick were laid flat making a wall 4 inches thick, and sand or concrete was filled in behind the wall as fast as the brick were laid. Unless this was done, the sunshine had a tendency to keep the asphalt so warm that the wall slid in some places and had to be rebuilt. When the brick were laid, it was found that there were a number of open joints, so before the final coat of asphalt was brushed over the surface, these joints were raked out and pointed with a rich mixture of asphaltic cement and sand much as a masonry wall is pointed with Portland cement.

STANDPIPES AND WATER TANKS.

Standpipes and water tanks are particularly applicable in two situations; one is where the surrounding country is so level as to afford no elevation suitable for a reservoir, and the other is where the amount of water to be stored is so small that the construction of a masonry basin is more expensive than the building of a suitable tank.

A standpipe is merely a large shell of wrought iron or steel plates put together much after the fashion of a boiler. The fundamental principles on which the design should be based are comparatively simple, and yet they have been violated frequently, and the last 25 years present a discreditable record of standpipe failures. A tank of this sort is subjected to wind pressure tending to buckle it, especially when empty, as was actually the case in a well-built structure near Providence. When the pipe is filled with water, it is subjected to a bursting strain in the metal and in the vertical joints, due to the pressure of the water within it. The wind pressure, in addition to its tendency to force in the plates on the side against which the wind blows, also strains the horizontal joints. A consideration of these strains is not a difficult problem. The stress in the metal tending to burst the pipe at any elevation may be found by computing the hydrostatic pressure of the water at that point, multiplying this pressure expressed in pounds per square inch by the diameter in inches of the standpipe, and dividing the product by twice the thickness in inches of the plate at that elevation. The result will be the stress per square inch due to the water at that elevation. The wind pressure is equal to about half the wind pressure on a flat surface of the same height and diameter as the standpipe; if it is assumed that the wind presses against a flat surface at about 45 pounds

per square foot, the pressure on a standpipe, expressed in pounds per square foot, will be 45 times the height of the pipe in feet multiplied by one-half its diameter, also expressed in feet.

It is now the established practice to use steel plates for such pipes, although for many years a number of prominent engineers refused to employ anything but a high grade of wrought iron. If steel is used it should be soft, open-hearth metal having an ultimate tensile strength of 55,000 to 62,000 pounds per square inch, with an elastic limit of half the ultimate strength. The minimum elongation in test pieces 8 inches in length should range from about 22 per cent. for plates less than $\frac{3}{8}$-inch thick to about 26 per cent. for plates more than $\frac{3}{4}$-inch thick. The cold-bend test should require the steel to be bent flat upon itself without fracture, except in the case of plates more than half an inch thick, when the bending should be done around a mandrel of a diameter equal to the thickness of the plate. It is probable that the best results would be obtained by requiring the sheared edges of the plates to be planed before the holes are punched, and to ream the holes before the plates are bent. This has never been done, so far as the writer knows, but it would certainly tend to prevent the use of plates having any defects along the edges, which weaken the standpipe in its most vulnerable point.

The best current practice in workmanship at the time of writing is shown by the following extracts from specifications prepared in 1897 by Mr. Nicholas S. Hill, Jr., at that time chief engineer of the Baltimore water-works: "The plates and angles shall be shaped to the proper curvature by cold rolling. No heating and hammering shall be allowed for straightening or curving. The work shall be carefully and accurately laid out in the shop, and the rivet holes punched with a center punch, sharp and in perfect order, from the surface to be in contact. The diameter of the punch shall not exceed that of the rivet by more than 1-16 inch, and the diameter of the die shall in no case exceed that of the punch by more than 1-16 inch. Rivet holes in plates having a thickness of $\frac{3}{4}$-inch and over shall be drilled and sharp edges shall be trimmed. All calking edges shall be planed to a proper bevel. All parts must be adjusted to a perfect fit and properly marked before leaving the shop. In assembling the work the rivet holes shall match so that hot rivets may be inserted without the use of a hammer. Drifting is prohibited. Eccentric holes, if any, must

be reamed, and if required larger size rivets shall be used in such holes. The best grade of soft charcoal iron rivets in the market shall be used. Sufficient stock must be provided in the rivets to completely fill the holes and make a full head. The rivets shall be driven at such a heat as will admit of their being finished in good form, with a button set before the rivet has cooled to a critical point. As often as may be deemed advisable for the purpose of testing the work, rivets shall be cut out at the direction of the inspector. The quality of the rivet metal and of the workmanship shall be such that the fracture of the rivet so removed at random shall show a good, tough, fibrous structure without any crystalline appearance, and there shall be no evidence of brittleness. Loose rivets must be promptly replaced, no rivet calking being permitted. All seams must be calked thoroughly tight with a round-nose calking tool by workmen of acceptable skill. Great care must be taken not to injure the under plate." Good painting is, of course, insisted on.

The method of computing the strength of riveted joints is too far removed from the scope of this book for discussion here; it is explained in Trautwine's Pocket-Book, which, as before stated, should be in the office of every water-works. The riveting never develops the full strength of the plates, as is shown by the following table prepared by Mr. Freeman C. Coffin:

THE STRENGTH OF RIVETED JOINTS—THE LETTERS REFER TO FIGURE 46.

	Thickness of plate.	Rivets.		Distance from edge.	Distance bet. centers.	Diagonal distance.	Percentage of strength of whole plate.
		Diam.	Length.	D	E	F	
Single rivet.....	¼	⅝	1¼	1	1⅛	.....	54.5
" "	⅜	⅞	1⅝	1	1⅛	.....	52.5
Double "	¼	⅝	1⅝	1	2¼	2	66.0
" "	⅜	⅞	1¾	1⅛	2⅝	2⅛	66.0
" "	⅜	⅞	2	1¼	2¾	2¼	64.7
" "	½	1⅛	2¼	1⅜	2⅞	2⅞	63.0
" "	½	1⅛	2½	1⅜	2⅞	2⅞	63.0
" "	⅝	1⅜	2½	1⅝	3	2⅛	62.0
" "	⅝	1	2¾	1⅝	3	2¾	62.0
" "	¾	1⅛	3	1⅝	3⅛	2⅞	61.0
" "	¾	1⅛	3¼	1¾	3¼	3	60.0
" "	⅞	1⅛	3⅜	1¾	2⅞	2⅞	61.0
" "	⅞	1⅛	3½	1¾	2⅞	2⅞	59.3
" "	1	1⅛	3¾	1⅞	3	2¾	59.0
" "	1	1¼	4	1⅞	3⅛	2⅞	59.0
Triple "	1	1⅛	3¾	1¾	4	3½	70.0

It is customary to use a factor of safety of five in figuring the metal of standpipes, which is equivalent to a factor of safety of

about three in the joints. As the weakest point of a structure determines its strength, it is evident that every precaution should be taken with the riveting. Since about 1893 the vertical seams have frequently been made butt joints with straps inside and outside. This is a decided advantage in securing good workmanship where the plates are over half an inch in thickness. Butt joints are always used in the bottom (horizontal) plates. Machine work gives better results than hand riveting, and the rivets should be heated on an elevated stage near the work. The best practice is to make the horizontal courses of plates alternate inside and outside, rather than have each course slightly tapering.

The top plates of an exposed standpipe are not usually less than ¼-inch thick, although thinner metal will withstand the bursting pressure. The additional thickness gives stiffness to the pipe and

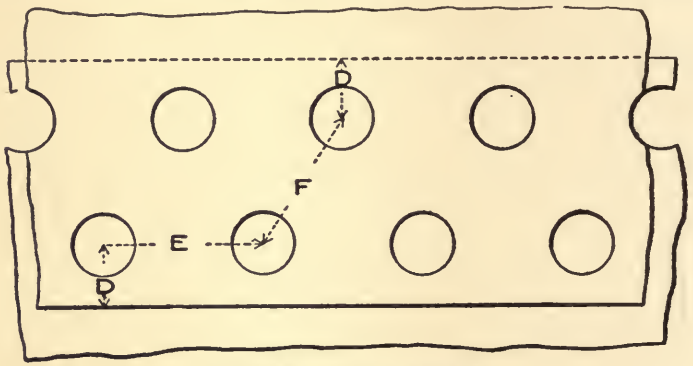


FIGURE 46.—RIVET DIAGRAM.

makes the calking comparatively easy. The top of the pipe should be strengthened by a ring of 3x3-inch or larger angle iron riveted outside the rim. The plates of the bottom are subjected to little stress but have to be made about three-fourths the thickness of the lowest vertical course in order to be calked well and to resist corrosion on their lower surface. The writer believes the bottom plates should project beyond the lowest ring course about 6 inches so that a 6x6-inch angle can be double-riveted to both around the outside of the tank. The horizontal leg of this angle can be bolted to the masonry foundation at as many points as seems necessary so as to avoid the use of brackets riveted to the side plates and anchored by rods to the foundation. These brackets

are used for preventing the overturning of the tank by wind pressure when it is empty, but they weaken the side plates and the bottom outside angle will answer the purpose just as well except when the tank is tall and slender, in which case it must be guyed by stays running to well-set posts.

The masonry foundation of a standpipe deserves particular attention, as several failures of these structures have shown. Concrete can generally be employed to advantage, but no matter what material is used it must be carried down far enough and spread wide enough to ensure absolute freedom from settlement of any sort. The bottom plates of the tank are generally put together on supports a few feet above the top of the masonry, which is then covered to a depth of 2 inches with a dry mixture of equal

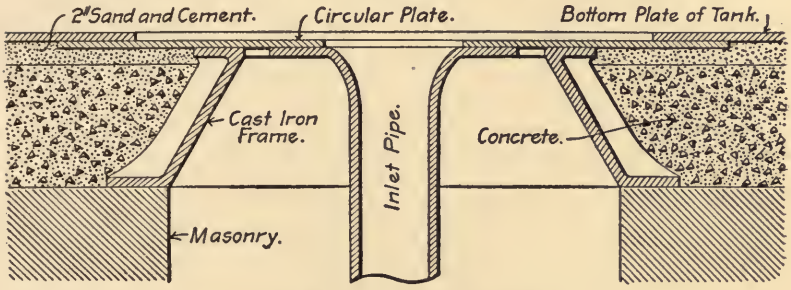


FIGURE 47.—INLET TO STANDPIPE.

parts of cement and sand. This affords a bed on which the bottom is lowered and makes it certain that the pressure on the masonry is uniform and the plates subjected to no excessive localized strains.

The pipe through which the water enters the standpipe is now generally a branch from the force main and serves also as the discharge main, the water level in the tank rising and falling as the demand for water fluctuates one way or the other from the uni form delivery of the pumps. The pipe is carried in an arched passageway through the masonry until it is about 6 feet inside the line of the shell and ends in a special quarter bend having a flange by which it can be anchored to a block of masonry. From this bend a pipe rises vertically and enters the tank by some such arrangement as is shown in Figure 47 or by a stuffing box and gland of the type used to pack the piston rods of steam cylinders. The overflow pipe should never be placed inside a tank in which ice

will form, for it will be ruined sooner or later. A telemeter, of which there are several types on the market, can be purchased at a low cost, and its use will enable the engineer of the pumping station to slow down his pumps whenever the standpipe is full. If an overflow pipe must be used, it should be placed outside the tank, as has been done in many instances, particularly when the tank is enclosed.

If the standpipe is on a plain, it is customary to provide the pipe connecting it with the force main with a valve which can be closed from the pumping station when a fire alarm is sounded. If this is not done, it is impossible to put sufficient pressure on the mains to obtain good fire streams. If the standpipe is situated on an elevation which, in itself, affords ample fire pressure, no such valve is needed. There are a number of such devices on the market, some worked by electricity, some by water pressure, and some automatically by the pressure in the pipes. A pressure-regulator such as is used between high-service and low-service street mains might be arranged to do this work, although the writer has no personal knowledge of such an installation.

A peculiar form of standpipe was built about 1890 at Fort Smith, Ark., from the plans of M. M. Tidd. It is 30 feet in diameter and 100 feet high. The pipe which connects it with the force main is 16 inches in diameter and branches just outside the masonry foundation into two pipes. One of these ends in a riser terminating in the bottom of the tank and is provided with a check valve which permits only a flow from the tank. The second branch is connected with a 16-inch pipe rising vertically through the center of the stand-pipe to an elevation of 6 feet above its top. This vertical pipe is guyed to the shell of the tank by half-inch wire rope with turnbuckle adjustments. The small pipe is filled by the first strokes of the pump and gives a full head to the supply unless the draft exceeds the delivery of the pump, in which improbable event the supply would be augmented by the water stored in the standpipe, the height of which would govern the pressure, as it does whenever the pump is not working. The standpipe can be filled only through water overflowing the top of the center pipe, and the considerable distance that the overflow falls retards the formation of ice.

By using a winding stairway and a conical roof it is frequently possible to make an uncased standpipe a fairly pleasing object.

The tank shown in Figure 48 was built a number of years ago for the Des Moines Water Company from the plans of Mr. Chester B. Davis. It is 30 feet in diameter, 100 feet high and furnished with a balcony and roof which relieves the hard lines of the tower. The

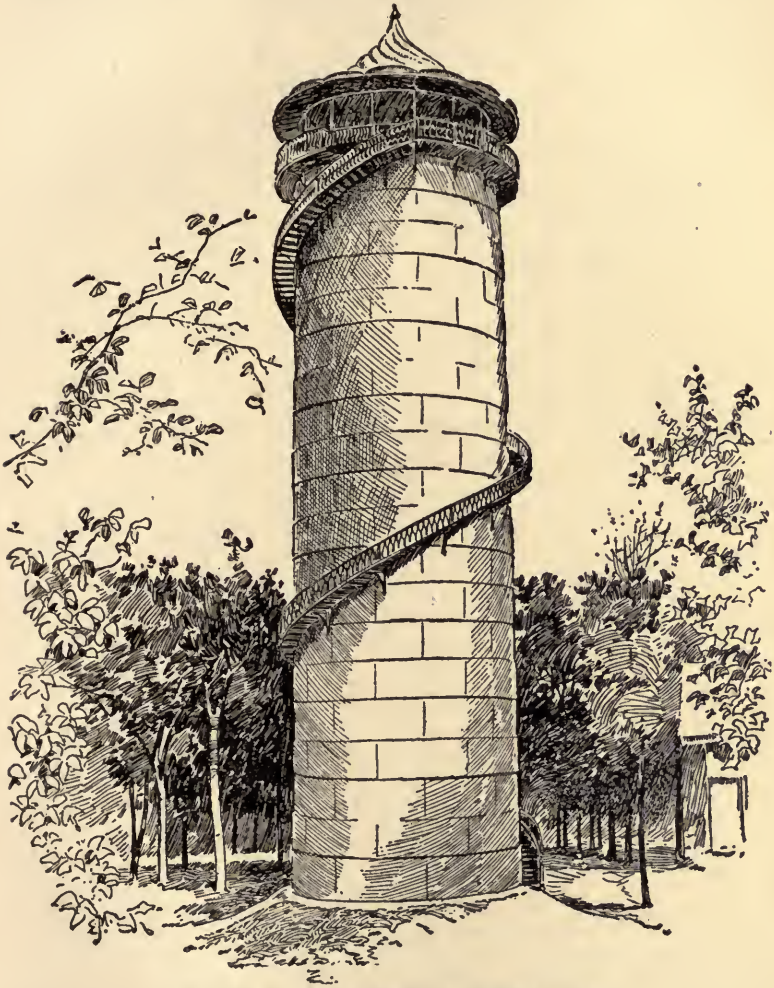


FIGURE 48.—THE DES MOINES STANDPIPE.

roof is covered with unpainted No. 22 sheet copper. The floor is supported by iron beams and consists of a 2-inch bottom course of pine finished with a course of selected 1-inch hard pine. The ceiling of the roof is No. 20 corrugated iron. Another attractive

FIGURE 49.—WATER TANK AT NORWOOD, OHIO.

tank, shown in Figure 49, was built at Norwood, Ohio, from the plans of Mr. G. Bouscaren.

It is sometimes desirable, occasionally even necessary, to enclose the standpipe in masonry to protect it from wind pressure and the cold. When this is done very attractive architectural effects can be produced, as is shown by completed structures and the results of a prize competition instituted by "The Engineering Record" in 1890. The best of the plans received in that competition are published by the journal mentioned. The masonry tower at Lawrence, Mass., is shown in Figure 50. The first 27 feet of the masonry above ground is broken granite ashlar, and the remainder is brick with granite trimmings. The main tower is octagonal in plan and there is also a projecting circular tower of 6 feet inside diameter in which an iron stairway winds upward to the balcony. The tank is 30 feet in diameter and 102½ feet high. The floor of the balcony is about 107 feet above ground, and the point of the roof about 157 feet.

FIGURE 50.—LAWRENCE WATER TOWER.

Where a town is located in a flat country it is self-evident that the water in the lower part of a tank is of no use whatever, as a head of 50 feet is needed to give pressure amounting to anything. This fact has led to the erection of tanks on the top of metal towers or masonry shafts, and such structures often prove more economical than a standpipe. The construction of the tank itself offers no features of special interest save that the bottom is usually made conical when steel framing is used for a support, unless the

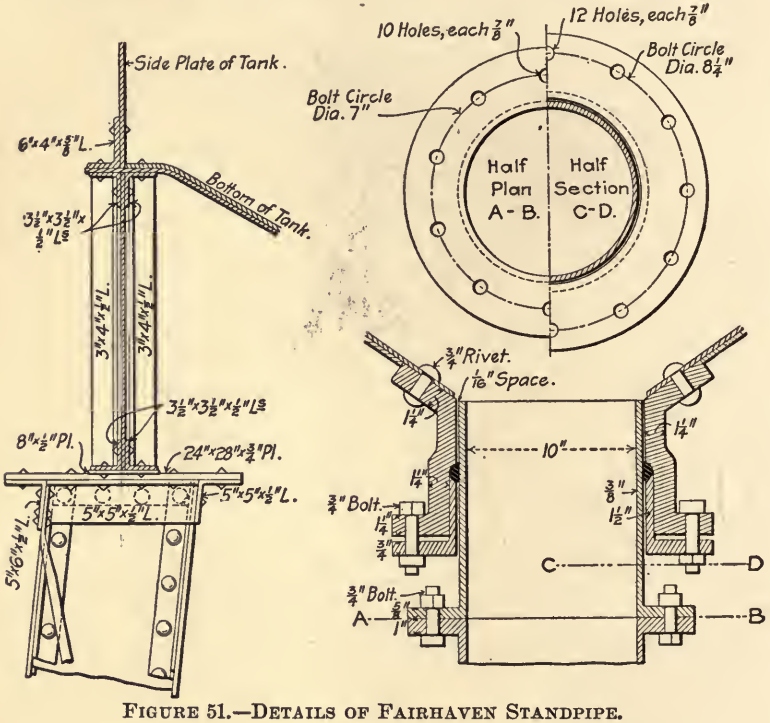


FIGURE 51.—DETAILS OF FAIRHAVEN STANDPIPE.

capacity of the tank is so small that it can be satisfactorily held on a platform resting on girders. If the bottom is conical, the supply pipe enters at its lowest point through a stuffing-box, as shown in Figure 51, which also gives details of the method of connecting the side and bottom plates and supporting the entire weight on a plate girder. These details are from a Massachusetts water tower designed in the office of the late M. M. Tidd by Mr. Freeman C. Coffin.

The design of the steel work for such a tower calls for an ac-

quaintance with subjects entirely foreign to this book, but it is believed that the detail drawings in Figure 52 will prove of value to many of its readers. This tank was designed by Mr. Edward Flad for the water-works in Laredo, Tex,. and is one of the few in this country with a curved bottom, although the pattern is a favorite abroad.

The water tower at Oberlin, Ohio, built from the plans of Mr. W. B. Gerrish, affords an instance of a masonry pedestal for a steel tank. It consists of a stone base 40 feet high above the ground, on which rests a steel tank 35 feet high and 18 feet in diameter. The concrete bottom course of the foundation of the base is 2 feet thick, and forms a ring 9 feet wide on which are four courses of rubble masonry having a total depth of 6½ feet, capped by a water table 6 inches thick. On this foundation the base was built, consisting of a light-colored sandstone tower 22 feet in exterior diameter at the bottom and 20 feet just below the coping. The walls are 3 feet thick at the bottom and 2 feet at the top, the batter being entirely on the exterior. The interior of the base is divided into three stories, two of 14 feet and a top one of 10 feet. Entrance to the tower is through a doorway measuring 3 x 7 feet, with a semi-circular glazed transom above it. The second floor is lighted by a round window 3 feet in diameter, and on the top floor there is a door furnishing access to a light iron balcony from which a ladder leads to the tank above. All masonry is quarry faced ashlar in regular courses, decreasing from about 24 inches at the bottom to 10 inches at the top. At least one-fourth of the masonry was required to be headers and at least four stones in each course had to extend entirely through the wall. The headers had to be two-thirds the thickness of the wall, and the stretchers had to be twice their height and of a width equal to their height at least.

EQUIVALENTS FOR STANDPIPES.

The purpose of a service reservoir or standpipe is two-fold; to furnish storage for a certain volume of water, and to furnish a head or pressure. Where no such works are built, the pumping plant must be run continuously at fluctuating rates and is subject to severe strains whenever hydrants are turned on or off or a pipe breaks. Several devices have been employed to remedy such shocks and there is a special system of works which provides a substitute for all functions of the standpipe.

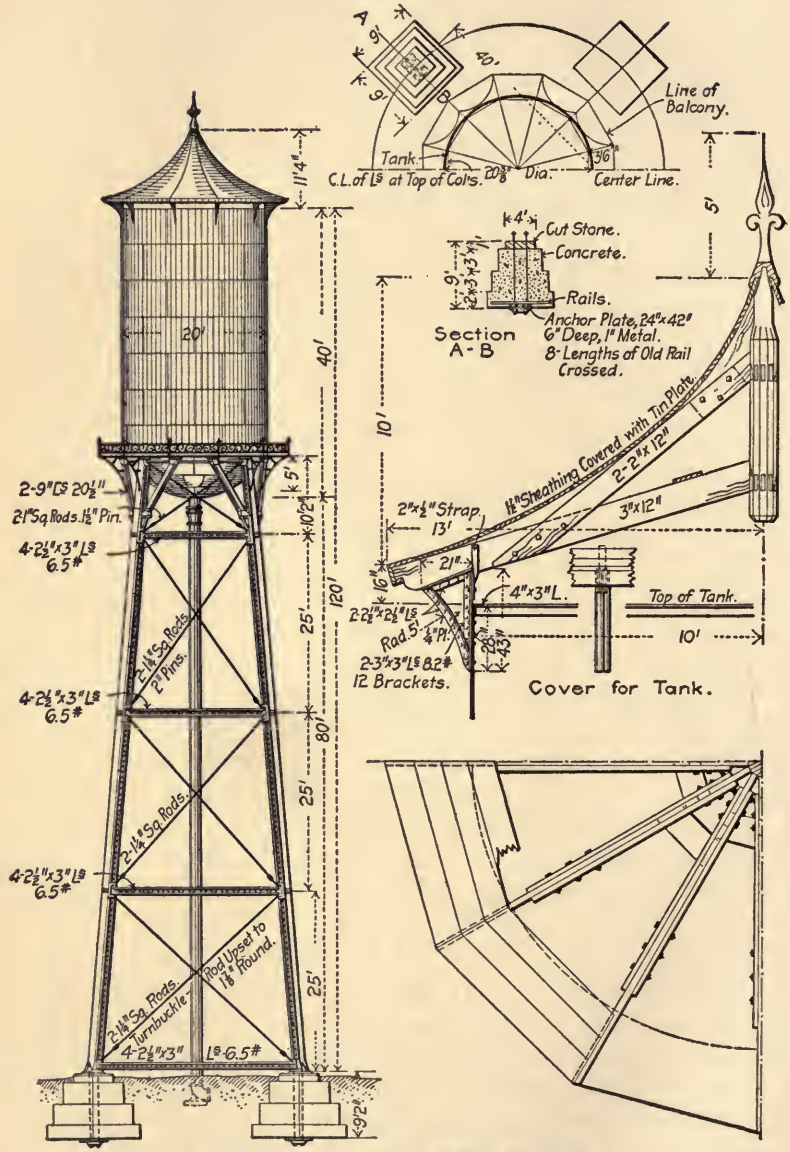


FIGURE 52.—LAREDO WATER-TOWER.

In the smallest direct-pumping European water-works an accumulator is occasionally employed. This is a strong vertical pipe with a stuffing-box at the top through which a heavily weighted

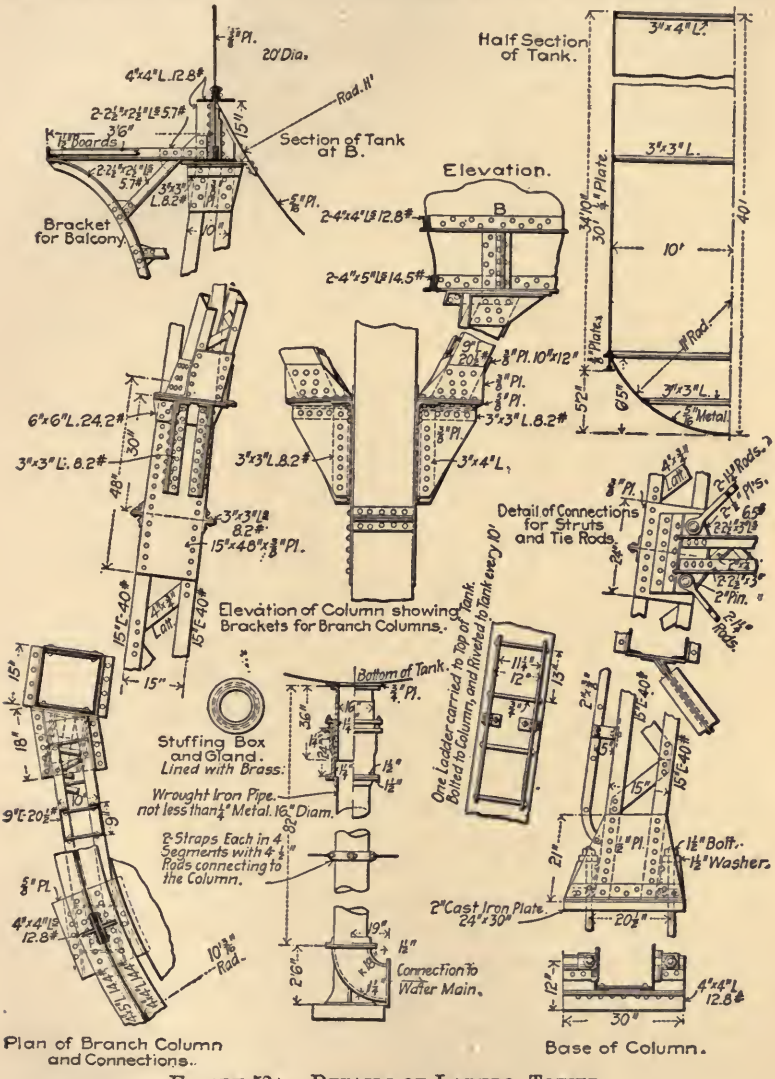


FIGURE 52A.—DETAILS OF LAREDO TOWER.

ram passes. The pipe is connected with the force main, and any marked fluctuations of pressure in the latter are prevented by the

movement of the ram. It is questionable if such an arrangement would prove satisfactory in more than a very few cases in this country.

Large air chambers on the force main are frequently used in Germany as a substitute for the pressure-equalizing function of the standpipe. In Herr Lueger's "Wasserversorgung der Staedte" the following outline of their theory is given. If V is the volume of air in an air chamber when the delivery of the pumps just equals the demand for water and P is the pressure of the air at this moment, then, if the demand for water varies by an amount, q, the air in the chamber has a new volume, v, corresponding to a changed pressure, p. Hence:

$$v - V = q \qquad pv = PV$$

The second equation is Mariotte's law. Combining the equations gives

$$p = P\frac{V}{V+q}$$

It is evident that the more the value of the second fact on the right-hand side approaches unity, the nearer become p and P. These pressures correspond, however, to heads of h and H feet in the force main and by substituting these values and altering the form of the equation the following relation will be obtained:

$$V = \frac{q\,h}{H-h}$$

By assuming various values of q and h the size of the air chamber can be easily determined for any given conditions.

One of the most interesting applications of these chambers with which the writer is acquainted was made many years ago in Augsburg. The engine room floor is about 33 feet below the highest street surface in the city, where a head of nearly 100 feet was desired. Between these points the loss of head due to friction was about 41 feet, so that the total dynamic head at the pumping station had to be about 174 feet. To elevate a tank to such a height would be very costly, so four air chambers were employed instead. Each is 32.8 feet high, 4.9 feet in diameter and has a capacity of nearly 800 cubic feet. The manner in which they are charged with air the writer has never succeeded in learning.

The air chamber of the Madison, Wis., works was designed by Mr. Edwin Reynolds to prevent the severe shocks on the pumping machinery forcing water directly into the mains, which were due to the sudden closing of hydrants. These water rams some-

times amounted to 40 pounds. The chamber is 5 feet in diameter, 20 feet high, and built of steel plates, and is on the main discharge pipe. At ordinary pressures it is filled with air. During a rise in pressure from 75 pounds, the ordinary amount, to 150 pounds for fire service, the chamber takes in about 200 cubic feet of water, and the expansive force of the compressed air promptly returns this water to the main in case of any sudden demand. In case a hydrant is suddenly closed, the chamber is again able to take up the excess of water while the engines are slowing down. A 2-inch feed pipe and a small air pipe run to a small receiver which has a set of valves so designed that it can be relieved of the pressure in the air chamber and the water exhausted from it, when it will fill with air. By changing the position of a lever, the air contained in this reservoir will then be forced into the air chamber by the water flowing from the latter into the former. This apparatus has been very satisfactory.

The experience of the Wyoming, Ohio, water-works in 1898 shows the value of such a chamber even on small plants. The pipes of these works were originally laid in an inferior manner, which resulted in much annoyance and expense due to the frequent blowing out of the joints, especially on the 10-inch force main and the lateral pipes running from it. During 1898 there was placed on the force main over the check valve at the pumping station, an air chamber 10 inches in diameter and 10 feet high to assist in cushioning water rams on the mains due to the sudden opening and closing of cocks and valves on domestic service pipes. Owing, however, to the lack of convenient facilities for charging the chamber with air, this was only partially successful. "To overcome the defect," a report of the Trustees reads, "we have placed a glass gauge on the air chamber and a small crank and flywheel air pump in the engine room, by means of which the air chamber may be conveniently charged to a fixed level and a good air cushion be always provided to receive the shocks on the pipe system. The beneficial effect of this is shown by the charts taken from the recording gauge at the pumping station. Heretofore, the rams on the pipe system would range from 20 to 40 pounds on the gauge and four or five might be noticed in an hour, while with the charged air vessel the rams are confined within 10 pounds and the air cushion is now so sensitive as to respond to all minor fluctuations of pressure on the pipe system."

Accumulators and air chambers do not afford storage and are poor substitutes for standpipes in consequence. This objection does not hold true of the system of compressed air and water tanks in use at Southampton and Babylon, N. Y., and several other places, which may be used whenever a standpipe is undesirable. The system was the joint invention of the late William E. Worthen and Mr. Oscar Darling and has a number of strong points, particularly where the water supply is drawn from wells and must not be exposed to the light before delivery to consumers.

In its main features the system involves the use of closed steel water tanks of any desired capacity into which the supply is pumped, closed steel air tanks into which compressed air is forced by a compressor in the pumping station, and a system of pipes and valves connecting the two sets of tanks. At Babylon, there are four water tanks with a combined capacity of 100,000 gallons, and two air tanks constructed to carry a maximum pressure of 150 pounds. Under ordinary conditions the reducing valve on the pipe between the air and water tanks keeps the pressure on the latter at 40 pounds, but in case of fire the full pressure of the air is thrown on the water tank and the supply is delivered under good fire pressure. All the tanks are underground in an extension of the pumping station, and the operating valves can be reached at once by the engineer. The system presents advantages well worth study, but these and the details of operation are so well stated in the trade literature of the people controlling the system that it is unnecessary to go into further explanation here.

CHAPTER XIX.—THE QUANTITY OF WATER TO BE PROVIDED.

There are probably more mistakes made in the estimates for the volume of water needed by towns and small cities than in any other feature of water-works design. The assumption is frequently made that a liberal allowance, say 60 gallons, for the daily supply of each person in the community and a liberal allowance for the rate of increase in the population, are all the factors to be considered. Now a good fire stream takes water at the same rate as about 6,000 people using the water for domestic purposes alone, so a bad fire in a town of 10,000 people could be checked with difficulty if the water-works were designed on this erroneous basis. It is evident, therefore, that fire protection should play a very important part in the final selection of plans for works.

The amount of water used for all purposes outside of fire protection varies somewhat according to the character of the population. In a town with many detatched houses surrounded by lawns which need sprinkling in summer, 50 to 60 gallons per capita daily should be provided for a population of 20,000. In a mill village of 10,000 population where but one or two fixtures will be used in a house, 40 gallons per capita may be more than enough. The amount needed for manufacturing purposes must be learned from a study of existing establishments, as industries like breweries or dyeworks require much larger supplies than machine shops or weaving mills. If there is plenty of water available it is well to see if any of the railways in the vicinity will buy water, for a considerable revenue may be obtained from supplies for locomotives if the price asked is a reasonable one. It is very important to provide for a large consumption, for there is sure to be some waste and a failure of a water supply, even if it is but partial, is a serious matter. In addition to the discomfort caused the people, and the unsanitary conditions which result, such a failure is

noticed on the fire insurance companies' records and tends to prevent the industrial development of the place.

It has been learned by experience that the draft during a few hours may be at twice the average daily rate per capita during the year. On Monday mornings, for instance, when washing is done, and late in the afternoons of hot days when lawns are sprinkled, there is an excessive demand for water, for which provision should be made.

FIRE PROTECTION.

In studying the fire protection which a water plant should afford a town, attention must be paid to the rules laid down by Mr. John R. Freeman, M. Am. Soc. C. E., in a valuable paper before the New England Water-Works Association. Pipes intended to supply water for domestic purposes only might start with main arteries and taper down to small veins at the border of the district. Fire protection, on the other hand, often demands a concentration of all the water the works can furnish at one point, which may happen to be almost anywhere. Moreover, there may be two fires to put out at the same time, and the works should be designed for such a contingency, although it would be unreasonable to design them for two great simultaneous conflagrations.

A stream from a hose does not put out a fire by wetting the flames. A fire involves, first, the roasting of the material so as to give off gases, and, second, the burning of these gases. The stream of water is intended to chill the burning materials so that no combustible gases are given off. If the fire is a strong one and the stream of water small, most of the water will be evaporated as it passes through the flames and but little reach the place where it is of any use. Hence, good substantial streams are needed, and feeble squirts from a small hose are of no value except in checking a fire as it begins. Large fire streams are of course out of the question in small cities, but it is well to remember that six streams from a 1-inch nozzle, each delivering 200 gallons a minute, are worth more in putting out a fire than ten $\frac{3}{4}$-inch streams of 120 gallons each. Both theory and practice have settled on the $1\frac{1}{8}$-inch smooth nozzle as the best general size for use, but it is probably too large for the class of works described in this book, except where large valuable buildings must be protected. A 1-inch stream will answer well for most purposes in a residence or suburban town and a $\frac{3}{4}$-inch stream will be sufficient for many places.

The afflictions that follow fires are too serious to be trifled with by adopting little streams. Mr. Freeman points out that a burning business block 50 feet square and three stories high demands just as many fire streams to extinguish it and to protect the buildings each side of it when it stands in a village of 2,500 inhabitants as when it stands in a city of twenty times that population.

The amount of water in a fire stream depends upon the size of the nozzle and the pressure of the water at its base. So far as the size of the nozzle is concerned, it is well to provide 250 gallons per minute for each 1⅛-inch nozzle, 200 gallons for each one of 1 inch, and 120 gallons for each one of ¾-inch bore. These quantities are based on a pressure of about 45 pounds at the base of the nozzle.

Whether such a pressure can be obtained depends upon whether good judgment has been displayed in building the works. Mr. Freeman's words are quoted verbatim, as they express the opinion of the leading authority on this subject in the United States.

"A 1⅛-inch, 250-gallon stream calls for a velocity of 16.34 feet per second in the hose. This velocity is far beyond what in other arts is regarded as the economical limit for the velocity of flowing water. In city water mains from 2 to 3 feet is the common velocity; in sewers the same is true; in the flumes supplying turbine water-wheels 3 to 5 feet can seldom be exceeded with propriety, and in the short delivery pipes to low-lift centrifugal drainage pumps, 10 feet per second is the common maximum.

"In fire hose we are held up to high, force-wasting velocities by the all important necessity of keeping the hose so small in diameter and weight that men can grasp it firmly, handle it easily, and move it around quickly. Practical experience has settled on 2½ inches internal diameter as the favorite size.

"To deliver 250 gallons per minute with only 3 feet per second velocity would require a hose or pipe nearly 6 inches in diameter. In other words you force as large a volume of water through a 2½-inch hose as would go through a 6-inch pipe at the 3-foot velocity common in water mains.

"It is true economy to be generous in the number of hydrants and thus save money on the outlay for hose and for making good its annual depreciation. Moreover, by the use of short lines of hose, there is a great gain in the efficiency of a stream by its increased force and greater volume and the greater height and distance to which it will reach.

"Good jacketed fire hose now costs about 75 cents per foot. A 6-inch tar-coated heavy cast-iron main can be laid for about 75 cents per foot, cost of pipe, trench, lead and laying all included. A city can buy a good two-way hydrant for less than the price of 50 feet of good fire department hose and its water department can buy and put down 100 feet of the best 6-inch cast-iron water pipe for just about the same price that its fire department pays for an equal length of hose. The life of the hose will not average more than 5 to 10 years. The pipes may last 50 years.

"The bed-rock facts on which our rule for spacing hydrants must rest are that a good stiff 1⅛-inch standard stream of 250 gallons per minute cannot be pushed through more than 400 feet of even the very best and smoothest hose by a hydrant pressure of 100 pounds.

"The water-works giving a hydrant pressure of more than 100 pounds are comparatively few and the liability of accident is so increased that it is as a general rule advisable not to exceed 100 pounds hydrant pressure. The average New England pressure is only about 75 pounds, therefore if one would use hose lines more than 300 feet long he must sacrifice the power or size and the efficiency of the jet, or use a steam fire engine to give it an extra push.

"This limit to the length of hose through which a good stream can be forced indicates that as a rule there should be hydrants enough around or near to any very important block of buildings in a city of moderate size without steamers and with 80 to 100 pounds hydrant pressure, so that eight hose streams could be led to it without the average line of hose being more than 300 to 400 feet in length, and with no one of these eight lines more than 500 feet in length. If the hydrant pressure is but 60 to 70 pounds, then the hydrants should be so placed with reference to any important building that half the whole number of streams could be drawn through lines of hose not exceeding 250 feet in length."

The application of these general principles to the local conditions of any particular town calls for careful study. Where the buildings are surrounded by lawns and a fire in one does not endanger others two streams are enough. As 175-gallon streams under 30 pounds nozzle pressure give fair protection for ordinary detached dwellings, a 500-foot line of 2½-inch hose may be tolerated if the hydrant pressure is 75 pounds or more. Four hundred feet is a much safer allowance, however, for the hydrant spacing

in such districts, and in the business portion of the town about 250 feet is the proper spacing.

There is little advantage in using four-way hydrants in the class of works under consideration. The two-way hydrants largely avoid the danger of a total lack of water from freezing, which is an important matter to consider. The position of the hydrants should be determined while on the ground and every detail of the surroundings can be taken in at a glance. If they are located from a map the result may be as surprising as in a plant once examined by the writer, which had a couple of hydrants at a place where the only structures within a radius of 500 feet were the tombstones of a large cemetery and a small open shed of rough boards used by a stone cutter.

Various tables have been prepared to show the number of streams which should be available, simultaneously in towns of different populations. As a basis of discussion among specialists they are of value, but they are liable to mislead people who have not given serious study to the subject. Their lack of general applicability may be readily understood by considering two typical villages of 5,000 people each. One is a town which contains few places of business and no factories; a suburban residence town, in fact. The other is a mill village, where the welfare of the entire community depends on a couple of factories. In the first case, from four to six streams are needed; but in the latter, ten or more strong 250-gallon streams may be demanded to protect the industries which are the sole support of the entire community. Ample allowance in this matter is all the more important because of the waste of water in case the service pipe of a burning building breaks. There have been cases, according to Mr. Freeman, where a small public water supply has been rendered utterly useless by the breaking, during the early stages of a fire, of a 3-inch or a 4-inch pipe entering a building.

In all water-works depending on an elevated tank or reservoir for water during the night or while reserve pumps are being brought into service, attention must be paid to the needs of fire protection in determining its size. The minimum supply which the Factory Mutual Insurance Companies request for the protection of large isolated mills is one hour's draft of the full number of fire streams. This may be taken as a satisfactory basis of figuring for small water-works on the direct-pumping system, where the

pumps can be started quickly, day or night. If, on the other hand, the reservoir is supplied by gravity or through a long force main, economy will probably result in most cases if the gravity conduit or the pumping station with its force main is proportioned for the maximum domestic draft, and the excess water is stored in a large reservoir holding at the very least six hours' supply for the maximum number of fire streams. As the maximum draft is about twice the average amount except in mill villages and other special cases, there will be little difficulty in obtaining the requisite volume of water for storage.

SIZE OF STREET MAINS.

The sizes of pipe to be laid in any street should be determined from the needs of fire protection in the first place, for if this is done it will generally be found that they are ample for delivering the domestic supply. The fundamental fact to be borne in mind is that small pipes cause a great loss in water pressure and a decrease in the efficiency of fire streams. The good influence of a large supply pipe, ample pumping capacity and abundant storage in an elevated reservoir or tank may be largely counteracted by small or badly arranged street mains and an insufficient number of badly located hydrants. The loss of pressure due to friction in clean, straight pipes 1,000 feet long, but of different diameters, when discharging 150 gallons per minute, is as follows:

Diameter of pipe, inches.....	4	6	8	10	12
Friction loss, feet.....	17.02	2.09	0.48	0.15	0.06

These figures give the most favorable results. After the pipes have been in use a number of years and their inner surfaces are roughened by tubercles which diminish their bore and increase their frictional resistance to the flow of water, it may easily happen that the resistances will be doubled, particularly when it is considered that there are bends and elbows in a line of street main. Under such practical conditions, a 4-inch pipe 1,000 feet long will require a head of 34 feet to deliver 150 gallons per minute, while a 6-inch pipe will need a head of less than $4\frac{1}{4}$ feet. It is very evident therefore that no reliance can be placed on 4-inch pipe for fire protection unless it is used in short lengths fed from both ends by larger pipes. It is useless to expect that a half mile of 4-inch pipe, tapped every hundred feet or so by domestic service pipes, will furnish at its end a stream large enough for any other use than washing carriages or windows.

On the other hand it is very easy to select street mains of too large size. No one who has not worked out a number of actual problems in street-main hydraulics can appreciate what differences in the cost of the system can be produced by variations in the arrangement of piping to obtain the same pressures. It will probably be found best, in attacking such problems, to prepare a rough sketch of the streets of the town, on which the hydrants and the relative elevations are indicated. The tables of fire streams accompanying this chapter will furnish data for determining the pressure at the hydrants when discharging streams of the desired character. The elevation of the hydrant above a given plane must carefully be considered, because it is much easier to furnish 75-pound pressure at the foot of a hill than at the top. Fortunately the greatest need of fire protection rarely occurs at the summit of hills. It seems hardly necessary to mention this point, yet most engineers have doubtless seen hydrants which furnish little more than a dribble because they are placed at elevations exceeding the available pressure on the distribution mains.

Having located the hydrants, a heavy fire should be assumed at some place. The hydrants within reach will throw two or more streams each. The street mains must now be proportioned so as to furnish the desired quantity of water under the necessary pressure and also the maximum quantity required for domestic service. A repetition of this process of computation in other parts of a town, care being taken every time to allow for the elevation of the hydrant as well as the pressure needed for the streams, will furnish an indication of the nature of the piping which is required. The remaining portions can be readily interpolated and it then becomes necessary to arrange for the connection of the street mains with the supply main. This is a question which must be solved independently for each town. The most unfavorable situation arises where a line of important buildings extends along a single street; the easiest problem to solve is where the buildings are fairly uniformly distributed on a network of streets, which can be threaded by lines of small pipe, possibly 4 inches in some places, which are fed by larger mains surrounding or intersecting them, as the case may be.

In communities of but a few thousand, these recommendations must be modified by the all-powerful restrictions of limited financial resources. It is in these small plants that the engineer has

an opportunity to exercise his common-sense and technical ability to the utmost. Every $100 additional cost must be carefully considered. In some places, such as the summits of hills or clusters of houses apart from the body of the town, it may prove best to recommend insurance rather than hose streams as a protection against fire losses. In any community it is the service of the whole and not the benefit of the individual which should be considered. If the financial resources of the town do not permit the construction of works which are of equal service to all, this is no reason for voting down such a desirable public improvement as a water plant for domestic service and fire protection for most of the homes.

FIRE STREAMS.

On account of the great importance of a thorough study of fire protection in designing water-works, a subject which usually re-

FIRE-STREAM DATA FOR ¾-INCH SMOOTH NOZZLE.
(This table also serves for a ⅞-inch ring nozzle.)

Indicated pressure, lbs.	Best fire jet.		Gallons per minute.	Hydrant pressure in pounds required to maintain pressure at base of stay pipe as per column 1, through 2½-inch hose lines mentioned.											
				50 ft.		100 ft.		200 ft.		300 ft.		400 ft.		500 ft.	
	Height, ft.	Reach, ft.		Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.
5	. .	.....	37	5	5	6	5	6	6	7	6	8	6	9	7
10	17	19	52	11	10	12	11	13	11	14	12	16	13	17	13
15	25	24	64	16	16	17	16	19	17	22	18	24	19	26	20
20	33	29	73	22	21	23	22	26	23	29	24	32	25	34	26
25	41	33	82	27	26	29	27	32	28	36	30	39	31	43	33
30	48	37	90	33	31	35	32	39	34	43	36	47	38	52	40
35	55	41	97	38	37	40	38	45	40	50	42	55	44	60	46
40	60	44	104	43	42	46	43	52	46	58	48	63	50	69	53
45	64	47	110	49	47	52	48	58	51	65	54	71	57	77	59
50	67	50	116	54	52	58	54	65	57	72	60	79	63	86	66
55	70	52	122	60	58	64	59	71	63	79	66	87	69	95	73
60	72	54	127	65	63	69	65	78	68	86	72	95	76	103	79
65	74	56	132	71	68	75	70	84	74	93	78	103	82	112	86
70	76	58	137	76	73	81	75	91	80	101	84	110	88	120	92
75	78	60	142	81	79	87	81	97	85	108	90	118	94	129	99
80	79	62	147	87	84	93	86	104	91	115	96	126	101	138	106
85	80	64	151	92	89	98	92	110	97	122	102	134	107	146	112
90	81	65	156	98	94	104	97	117	102	129	108	142	113	155	119
95	82	66	160	103	99	110	102	123	108	137	114	150	120	163	125
100	83	68	164	109	105	116	108	130	114	144	120	158	126	172	132

Eighty pounds per square inch is now considered the best hydrant pressure for general use; 100 pounds should not be exceeded except for very high buildings. If the nozzle is much higher or lower than the hydrant, allowance for difference of level must be made on hydrant pressure; 10 feet in height corresponds to 4½ pounds water pressure.

The above pressures are at the hydrant head while stream is flowing. The corresponding pump or reservoir pressure must be greater than the hydrant pressure by an amount equal to the friction loss between the hydrant head and pump or reservoir.

ceives inadequate emphasis in books on water-works construction, a number of tables are presented in pages 258 to 262 giving the results of experiments with fire streams made by Mr. Freeman and described by him in a paper of unusual value in the "Transactions" of the American Society of Civil Engineers for November, 1889. A study of these tables will give a better idea of the relations of the various conditions influencing fire streams than the reading of many pages of generalities on the subject.

It will be noticed that these tables are all for smooth conical nozzles. The reason for this is that the experiments demonstrate beyond question that this type is more effective than ring nozzles. The latter throw less water than a smooth nozzle of the same diameter, with the same hydrant pressure and hose; consequently the

FIRE-STREAM DATA FOR ⅞-INCH SMOOTH NOZZLE.
(This table also serves for a 1-inch Ring Nozzle.)

Indicated pressure, lbs.	Best fire jet.		Gallons per minute.	Hydrant pressure in pounds required to maintain pressure at the base of play pipe as per column 1, through 2½-inch hose lines mentioned.											
				50 ft.		100 ft.		200 ft.		300 ft.		400 ft.		500 ft.	
	Height, ft.	Reach, ft.		Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.
5	..	.....	50	6	5	6	6	8	6	9	7	10	7	12	8
10	18	21	71	12	11	13	11	16	13	18	14	21	15	23	16
15	26	27	87	17	16	19	17	23	19	27	21	31	22	35	24
20	34	33	100	23	22	26	23	31	25	36	27	42	30	47	32
25	42	38	112	29	27	32	29	39	31	45	34	52	37	59	40
30	49	42	123	35	33	39	34	47	38	54	41	62	45	70	48
35	56	46	133	41	38	45	40	54	44	64	48	73	52	82	56
40	62	49	142	46	43	52	46	62	50	73	55	83	59	94	64
45	67	52	150	52	49	58	51	70	57	82	62	94	67	105	72
50	71	55	159	58	54	65	57	78	63	91	69	104	74	117	80
55	74	58	166	64	60	71	63	85	69	100	75	114	82	129	88
60	77	61	174	70	65	78	69	93	75	109	82	125	89	140	96
65	79	64	181	75	71	84	74	101	82	118	89	135	96	152	104
70	81	66	188	81	76	90	80	109	88	127	96	145	104	164	112
75	83	68	194	87	82	97	86	117	94	136	103	156	111	175	120
80	85	70	201	93	87	103	91	124	101	145	110	166	119	187	128
85	87	72	207	99	92	110	97	132	107	154	116	177	126	199	136
90	88	74	213	104	98	116	103	140	113	163	123	187	134	211	144
95	89	75	219	110	103	123	109	148	119	173	130	197	141	222	152
100	90	76	224	116	109	129	114	155	126	182	137	208	148	234	160

Eighty pounds per square inch is now considered the best hydrant pressure for general use; 100 pounds should not be exceeded except for very high buildings. If the nozzle is much higher or lower than the hydrant, allowance for difference of level must be made on hydrant pressure; 10 feet in height corresponds to 4⅓ pounds water pressure.

The above pressures are at the hydrant head while stream is flowing. The corresponding pump or reservoir pressure must be greater than the hydrant pressure by an amount equal to the friction loss between the hydrant head and pump or reservoir.

friction loss in the hose is less and the stream is carried somewhat higher and farther, though of much smaller volume.

All the figures in the table are for 2½-inch hose, which is now the standard size. A slight variation in the diameter makes an important change in the friction loss. Using the same quality and length of hose to discharge a definite quantity of water, and designating the friction loss in a 2½-inch hose as 100, the loss in lines of different diameters will be about as follows:

Diameter	2½	2⅜	2¼	2⅛	2
Loss	100	129	170	200	305

In other words, it will require three times the head at the hydrant to produce the same stream with a 2-inch hose as with one half an inch larger in diameter. It seems hardly necessary to say anything further in favor of the large hose.

FIRE-STREAM DATA FOR 1-INCH SMOOTH NOZZLE.

Indicated pressure, lbs.	Best fire jet.		Gallons per minute.	Hydrant pressure in pounds required to maintain pressure at base of play pipe as per column 1, through 2½-inch hose lines mentioned.											
				50 ft.		100 ft.		200 ft.		300 ft.		400 ft.		500 ft.	
	Height, ft.	Reach, ft.		Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen	Rubber.	Linen.	Rubber.	Linen.	Rubber.
5		....	66	6	6	8	6	10	7	12	8	14	9	17	10
10	18	21	93	13	12	15	12	20	14	24	16	29	18	33	20
15	26	30	114	19	17	23	19	29	22	36	25	43	28	50	30
20	35	37	132	26	23	30	25	39	29	48	33	57	37	66	41
25	43	42	147	32	29	38	31	49	36	60	41	71	46	83	51
30	51	47	161	38	34	45	37	59	43	72	49	86	55	99	61
35	58	51	174	45	40	53	44	68	51	84	57	100	64	116	71
40	64	55	186	51	46	60	50	78	58	96	66	114	73	132	81
45	69	58	198	57	52	68	56	88	65	108	74	129	83	149	91
50	73	61	208	64	57	75	62	98	72	120	82	143	92	166	102
55	76	64	218	70	63	83	69	108	79	132	90	157	101	182	112
60	79	67	228	77	69	90	75	117	87	144	98	171	110	199	122
65	82	70	237	83	75	98	81	127	94	156	107	186	119	215	132
70	85	72	246	89	80	105	87	137	101	168	115	200	128	232	142
75	87	74	255	96	86	113	94	147	108	181	123	214	138	248	152
80	89	76	263	102	92	120	100	156	115	193	131	229	147	....	162
85	91	78	274	109	98	128	106	166	123	205	139	243	156	....	173
90	92	80	279	115	103	135	112	176	130	217	147	257	165	....	183
95	94	82	287	121	109	143	118	186	137	229	156	....	174	....	193
100	96	83	295	128	115	150	125	195	144	241	164	....	183	...	203

Eighty pounds per square inch is now considered the best hydrant pressure for general use; 100 pounds should not be exceeded except for very high buildings. If the nozzle is much higher or lower than the hydrant, allowance for difference of level must be made on hydrant pressure; 10 feet in height corresponds to 4⅓ pounds water pressure.

The above pressures are at the hydrant head while stream is flowing. The corresponding pump or reservoir pressure must be greater than the hydrant pressure by an amount equal to the friction loss between the hydrant head and pump or reservoir.

The two classes of hose mentioned in the tables are unlined linen and the ordinary best-quality rubber-lined grade with a smooth interior. What is known as mill hose gives results between these two; it is a rubber-lined cotton hose with a rough interior. The couplings and other fittings which have threads should be made to the gauge of some large city in the vicinity. Over 200 gauges are used in the country; a ridiculous condition which should be remedied by the adoption of a set of standards. If the town adopts the gauge of the nearest large city it will probably be able to get its supplies more quickly when they are needed than if it selects one of the sets specially designed by some engineer anxious to furnish something brand new for each plant he plans.

In using the tables it should be clearly understood that with nozzles of 1 inch or more diameter indicated pressures at the base

FIRE-STREAM DATA FOR 1⅛-INCH SMOOTH NOZZLE.

Indicated pressure, lbs.	Best fire jet.		Gallons per minute.	Hydrant pressure in pounds required to maintain pressure at base of play pipe as per column 1, through 2½-inch hose lines mentioned.											
				50 ft.		100 ft.		200 ft.		300 ft.		400 ft.		500 ft.	
	Height, ft.	Reach, ft		Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.
5	...	.....	84	7	6	9	7	13	9	16	10	20	12	24	13
10	18	22	119	15	12	18	14	26	17	33	20	40	24	48	27
15	27	31	146	22	19	27	21	38	26	49	31	60	35	71	40
20	36	38	168	29	25	36	28	51	34	66	41	80	47	95	54
25	44	44	188	36	31	45	35	64	43	82	51	101	59	119	67
30	52	50	206	44	37	55	42	77	52	99	61	121	71	143	80
35	59	54	222	51	43	64	49	89	60	115	71	141	82	166	94
40	65	59	238	58	50	73	56	102	69	131	81	161	94	190	107
45	70	63	252	65	56	82	63	115	77	148	92	181	106	214	120
50	75	65	266	72	62	91	70	128	86	164	102	201	118	238	134
55	80	69	279	80	68	100	77	140	95	181	112	221	130	262	147
60	83	72	291	87	74	109	84	153	103	197	122	241	141	.	160
65	86	75	303	94	81	118	91	166	112	214	132	261	153	....	174
70	88	77	314	101	87	127	98	179	120	230	143	....	165	....	187
75	90	79	325	109	93	136	105	191	129	246	153	....	177	...	201
80	92	81	336	116	99	145	112	204	138	263	163	....	188	....	214
85	94	83	346	123	106	154	119	217	146		173	....	200	....	227
90	96	85	356	130	112	164	126	230	155	....	183	....	212	....	241
95	98	87	366	138	118	173	133	242	163	...	194	....	224	....	254
100	99	89	376	145	124	182	140	255	172	....	204	....	236	...	....

Eighty pounds per square inch is now considered the best hydrant pressure for general use; 100 pounds should not be exceeded except for very high buildings. If the nozzle is much higher or lower than the hydrant, allowance for difference of level must be made on hydrant pressure; 10 feet in height corresponds to 4⅓ pounds water pressure.

The above pressures are at the hydrant head while stream is flowing. The corresponding pump or reservoir pressure must be greater than the hydrant pressure by an amount equal to the friction loss between the hydrant head and pump or reservoir

of the play pipe of 20 pounds or less give only feeble streams; 25 to 30 pounds will furnish a fair stream, 35 to 45 pounds a good stream, 50 to 60 pounds an excellent stream. With such nozzles, pressures of 65 pounds or more at the base of the play pipe make the work of directing the stream difficult without special apparatus. When nozzles under 1 inch in diameter are used, pressures less than 25 pounds will give streams of little use in extinguishing a fire which has a good start, although they may be of assistance in preventing its spread.

FIRE-STREAM DATA FOR 1¼-INCH SMOOTH NOZZLE.

Indicated pressure, lbs.	Best fire jet.		Gallons per minute.	Hydrant pressure in pounds required to maintain pressure at base of play pipe as per column 1, through 2½-inch hose lines mentioned.											
				50 ft.		100 ft.		200 ft.		300 ft.		400 ft.		500 ft.	
	Height, ft.	Reach, ft		Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.	Linen.	Rubber.
5	..	..	105	8	7	11	8	17	11	23	13	28	15	34	18
10	19	22	148	17	14	23	16	34	21	46	26	57	31	68	36
15	28	32	181	25	21	34	24	51	32	68	39	85	47	102	54
20	37	40	209	34	27	45	32	68	42	91	52	114	62	137	72
25	46	47	234	42	34	57	40	85	53	114	65	142	77	171	90
30	53	54	256	51	41	68	49	102	63	136	78	171	93	205	108
35	60	59	277	59	48	79	57	119	74	159	91	199	109	239	126
40	67	63	296	68	55	91	65	136	84	182	104	227	124	273	144
45	72	67	314	76	62	102	73	153	95	205	117	256	140	....	162
50	77	70	331	85	68	113	81	170	106	227	130	...	155	...	180
55	81	73	347	93	75	124	89	187	116	249	143	...	170	....	198
60	85	76	363	102	82	136	97	204	127	....	156	...	186	...	216
65	88	79	377	110	89	147	105	221	137	...	169	...	201	....	234
70	91	81	392	118	96	158	113	238	148	....	182	....	217	....	252
75	93	83	405	127	103	170	121	255	158	...	195	....	232	....	.. ..
80	95	85	419	135	110	181	129	...	169	....	208	....	248	....	....
85	97	88	432	144	116	192	137	....	179	...	221	....	...		....
90	99	90	444	152	123	204	145	..	190	.. .	234	....	....	....	....
95	100	92	456	161	130	215	154	....	201	...	247	...		...	....
100	101	93	468	169	137	226	162	....	211	..	261	...	....	....	....

Eighty pounds per square inch is now considered the best hydrant pressure for general use; 100 pounds should not be exceeded except for very high buildings. If the nozzle is much higher or lower than the hydrant, allowance for difference of level must be made on hydrant pressure; 10 feet in height corresponds to 4⅓ pounds water pressure.

The above pressures are at the hydrant head while stream is flowing. The corresponding pump or reservoir pressure must be greater than the hydrant pressure by an amount equal to the friction loss between the hydrant head and pump or reservoir.

CHAPTER XX.—THE WATER-WORKS DEPARTMENT.

In the preceding pages an attempt has been made to explain the principles which should govern the design and construction of a small water plant, but there remains for consideration the important subject of paying for and running the works. If the plant is the property of a private corporation, the stockholders have presumably undertaken the enterprise after full consideration of this subject from an investor's point of view. In this case the writer has no advice to offer. If the plant is built by a community, however, certain hints may prove useful.

So far as the financial problems are concerned, no definite plans can be laid until the state laws on the subject have been studied. In some states no bonds can be issued without a sinking fund to extinguish them on maturity, and the life of such bonds is frequently limited. In other states no charge can be made for water furnished for public purposes, which lets the entire expense fall on the consumers. It is therefore necessary to bear carefully in mind that the ideal system which is advocated in this chapter may be impracticable legally in some places in some respects, and must be modified so as to be as nearly equitable to everyone as the law permits.

An issue of bonds is made in most cases to cover the first cost of the works. The duration of these bonds should theoretically correspond in some degree with the life of the plant, and here it is seen at once that real estate, reservoirs, masonry conduits and such portions of the works have a term of existence to which the pipes, and still more the pumping machinery, cannot be compared. Moreover the works are commonly designed to meet the requirements of but a limited period of years, and it would be manifestly improper to issue bonds for a longer period than the assumed length of service of the plant. In a small community, the bonds are for such a comparatively small sum that they must form a

single issue, and their period should therefore be obtained by weighing all the influences mentioned. In large works it is preferable to follow the British practice of issuing long-period bonds for real estate and other permanent investments and short period bonds for the perishable portions of the undertaking.

After the plant has been put in operation, it is subject to a number of charges of different classes. The most important are: 1, the interest on the bonds; 2, the sinking fund for extinguishing the bonds at maturity; 3, the cost of ordinary repairs; 4, a charge for the depreciation of the plant; 5, the expense of extensions; 6, the operating expense. There are others which are of so little importance in the case of small works as to be of trifling significance. In raising revenue to meet these expenses it is important to consider the purpose of the works, the features which controlled its cost, and the uses of the water.

It has been shown in Chapter XIX. that the capacity of the works is very largely influenced by the demands of satisfactory fire protection. Estimates made by a number of engineers show that one-third of the total cost of a plant is generally spent to provide enough hydrants, ample mains and sufficient pressure to meet such requirements. This protection is afforded to the property of the place generally, and is recognized by insurance companies in establishing their rates. It is therefore just that a considerable portion of the revenue of the water-works department should be met by a general tax on account of this service.

A certain amount of water is also used in schools and other public buildings, for flushing sewers, sprinkling streets and similar purposes, all of which are for the general good of the community and should be paid by it. The amount of water thus used will run anywhere from 5 to 15 per cent. of the whole quantity. To this amount should be added the leakage from street mains, amounting to 2,500 to 3,000 gallons per mile of pipe daily in a well-built and carefully maintained plant. Since the town owns the plant this leakage should be charged to the town as such, rather than against the percentage of the people who may be consumers.

The plant has been constructed of larger capacity than the present demand for water and, roughly speaking, a quarter of its total cost has been expended in provisions for future needs. If this is taken into account in fixing the rates for water, these will be un-

duly high at first, when they should be as low as equity will allow in order to encourage the people to become consumers and as an inducement to industries to come to the town.

A water pipe in a street adds an appreciable amount to the value of abutting property, just as do improved pavements and sewers. On this account the writer believes that such property may justly be taxed for a certain proportion of the cost of the street mains; say an amount equal to the cost of a 4-inch main without hydrants. The extra cost of larger mains and of the reservoirs, pumps and other portions of the plant, and additions to them from time to time, should be considered part of the capital expenditure for the works as a whole.

The operating expenditures and ordinary repairs should be paid by all the consumers, public and private, since they are incurred in their behalf solely.

The provisions for sinking fund and depreciation are usually combined in this country, for the reason that people forget a sinking fund is for paying off a bonded indebtedness while a depreciation fund is for renewing a plant when it is worn out. It is marked injustice to the present generation to require it pay off in twenty or thirty years the bonds it issues for works and at the same time accumulate a fund for the construction of a new plant of which a future generation only will enjoy the benefits. In states where a sinking fund is mandatory, it is not just to raise a fund for depreciation. The result of such laws is to turn over to the new generation "a property largely free from debt, but in a more or less deteriorated condition; but that generation may use the available credit and taxing power of the city to renew the seriously deteriorated portions of the plant, to extend the water service, and to enlarge slightly or reinforce the more substantial portions of the property in anticipation of probable future requirements." In states where sinking funds are not required by law, a real depreciation fund may be created and the old bond issues taken up by new ones. In this way the water department always has to its credit funds for new pumps and pipes with which to replace those worn out in service.

In one or two states, water departments are obliged by the courts to make a uniform charge for water, whether it is used in large or small quantities. The injustice of this is beginning to be generally understood. It is based on the assumption that it costs the de-

partment the same to supply a large quantity of water to a single consumer as to many smaller ones, each of which requires as much book-keeping and meter reading as the one large consumer. The problem of adjusting these charges on an equitable basis has been solved by what is known as the Madison schedule, devised by Mr. John B. Heim, superintendent of the Madison, Wis., water department. In this schedule, the bills of each consumer are made out every six months as follows: First 5,000 cubic feet, 20 cents per 100 cubic feet. Over 5,000 and up to 20,000 cubic feet, $10 for first 5,000 cubic feet and 10 cents for each additional 100 cubic feet. Between 20,000 and 30,000 cubic feet, $25 for the first 20,000 cubic feet and 5 cents for each additional 100 cubic feet. The schedule runs much higher, but the charges are made in the same manner. The minimum charge per six months is $2.25.

A caution should be given against the very common but very bad schedule which reads about as follows: For quantities under 5,000 cubic feet, 20 cents per 100 cubic feet. For quantities between 5,000 and 10,000 cubic feet, 10 cents per 100 cubic feet, etc. Such a schedule is a direct invitation to intentional waste under certain circumstances. If a consumer finds just before the meter reader is due that he has taken 4,000 cubic feet, costing him $8, it will be to his advantage to leave his fixtures open until 5,000 cubic feet are registered; for if the meter reader finds the consumption has been 5,500 cubic feet the bill will be but $5.50. No such juggling is possible with the Madison schedule, and it is not surprising to find it growing rapidly in favor. The actual charge to be made per 100 cubic feet or gallons depends, of course, on the cost of the water to the department and the percentage of the total expense of the works which are met by the water rates.

CHECKING WASTE.

When water-works were first built in this country, there was comparatively little difference between the various houses of a town and the water rates were made uniform on each building or dependent on the number of people it sheltered, or on its height and frontage. Such a plan answered fairly well for a time, but it was finally learned by observation that there was a difference in the amount of water taken in houses holding the same number of persons. The next step was to make the charge for water dependent on the number and variety of the fixtures. This was unquestionably a step forward, for it was then an easy matter to ascer-

tain roughly the amount of water probably taken for legitimate purposes from each type of fixture, for breweries and other manufacturing purposes, masonry, etc. In this way schedules were prepared from which charges were made out with somewhat nearer approach to justness than by the primitive methods.

There was a marked difference in the quality of fixtures after a time, and in the workmanship of the plumbers who placed them in the buildings. Some fixtures were found to be very wasteful of water; the total waste became so great as to make it necessary to build new works long before they were really needed, and in some cities the pressure on the street mains was inadequate for fire protection. The trouble was the same in Great Britain as here, but the methods of overcoming it which were first introduced in the two countries differed considerably.

The house-to-house inspection practiced in the United States and Great Britain proved fairly successful in small communities in determining the number of fixtures and their condition at the time of the inspector's visit. It did not accomplish very much, however, because a poor fixture would not be transformed into a good one just because a representative of a water department looked at it, nor were the occupants of the house always as careful to keep the water from running to waste as when an official was paying his visit.

The remedy for poor plumbing adopted in Great Britain was to require all fixtures to pass an official test and be stamped before they could be used, and even then they had to be placed in accordance with prescribed rules. This plan is so far at variance with American character that it was never introduced into the United States, to the writer's knowledge, although it proved a comparative success in Great Britain. It is described in detail in the book entitled "Water-Waste Prevention," written by Mr. Henry C. Meyer nearly twenty years ago, after a personal study of the results accomplished by it. This book is still the standard authority on water waste subjects.

It is probable that the importance of preventing the waste of water was first brought prominently before the American public by this work, which appeared originally as a series of articles in "The Engineering Record." A few American cities, notably Providence, had undertaken to check waste by the use of meters, but these devices were then in their infancy, they were compara-

tively expensive, and people who had become accustomed by the older methods of distribution to regard water as something nearly free as air, raised an unjust opposition to their use.

The extensive introduction of water closets, the increased variety and number of fixtures in houses, and the manufacture of the flimsiest sort of faucets and cocks soon made it apparent that some means had to be taken to prevent waste. House-to-house inspection was only slightly successful as a rule, and the practice of testing and stamping fixtures could not be introduced here, even were it desirable, on account of trade opposition too powerful to overcome. The consequence was the adoption of regulations in many cities authorizing the water department to place meters on the service pipes of each building where waste continued after the occupants had been notified to check it. The general passage and enforcement of these regulations gave an impetus to the manufacture of meters, old types were improved, new ones developed, and the result is that to-day the sale of water, like that of gas, can be carried on justly only by metering the services. This is now definitely determined by actual experiments, of which but one of many need be mentioned here. It is stated exactly as described in the annual report of the Lowell, Mass., Water Board for 1894.

"A, B, C, etc., represent certain pieces of property in Lowell. On this property (paying annual rate charges at the time), and unbeknown to owner as far as possible, a meter was attached during the summer months. The quantity of water actually being used

		Estimate, 1 yr. on 45 days' run. Cubic feet.	Present yearly rate paid.	Value of water by meter.	Price per 100 cu. ft. being paid city.
A.	5 tenements.....	24,000	$44.00	$33.60	$0.183
B.	12 "	34,200	51.00	47.88	0.149
C.	4 offices, 2 halls, 3 stores	116,250	79.00	174.37	0.067
D.	5 tenements.....	86,025	57.00	120.40	0.066
E.	3 "	18,000	26.00	25.20	0.144
F.	9 "	53,375	73.00	74.72	0.136
G.	12 "	176,011	123.00	246.42	0.069
H.	16 "	58,400	93.00	81.76	0.159
I.	3 "	49,125	31.00	68.77	0.063
J.	4 "	107,877	40.00	151.00	0.033
K.	5 "	139,200	40.50	194.88	0.030

was thus ascertained. From this fact and the yearly rate paid, the results in the table were obtained."

The last column contains the pith of the whole matter. Because of the absolute inaccuracy of any schedule based on the number and class of fixtures, the city was being paid 3 cents per 100 cubic feet by one man, and 18.3 cents by another, or more than six times as much. This was an injustice between consumers. The city itself was sometimes paid more, but generally very much less than the value of the service it rendered.

These figures of actual results, easily supplemented by many more of the same tenor, explain the value of meters in checking waste. Some people will be careful because they are intelligent and understand that carelessness and degeneration go hand in hand, others are careful because they wish to save the expense of carelessness, and others will not be careful under any conditions. If water is not supplied by meter the careful people have to pay for their own supplies and most of the waste of the others; if meters are used, the first two classes pay for only the water they use and the last class pays for what it uses and wastes, just as it should do. The cost of operating the plant is kept down, street main pressures are kept up, and water can be supplied at a low price. Let any other system be introduced and waste will ensue, operating expenses mount up rapidly, fire protection will become difficult, and the charges for water will be high. It is very probable that the general introduction of meters to-day in cities where they are not used, would check waste to an extent more than paying for their cost in a few years.

The statement that the universal use of meters in a small community is prohibited by their cost is not true. If water is sold from the outset by meter measurement the careful habits which soon become second-nature result in the prevention of enough waste to save in operating expenses in a few years the cost of the meters. It is probable that much of the trouble attending the introduction of these appliances has been due to the conditions of their use. Instead of their being purchased by the consumer it will be found more satisfactory for the water department to buy them in quantities. An advertisement in "The Engineering Record" asking for bids on lots of a hundred or more meters will be read by every manufacturer in the country and probably result in placing a contract at a considerably lower figure than can

be obtained by buying a few meters at a time, as needed, through a local plumber.

The water department should charge each consumer about one dollar annually for the setting and use of the meter, including repairs and renewal when it wears out. This sum is just to the consumer and the department, and keeps the control of the meters in the hands of the city. In case a consumer wishes to have a meter tested, it should always be done provided he signs an agreement to pay the cost of the work if the meter is found to be accurate or to register less than the amount of water which flows through it.

So far as the writer has been able to ascertain but one attempt was ever made to ascertain on a large scale the amount of water used by various fixtures. This was done at Newton, Mass., by Mr. J. Whitney, and reported to the New England Water-Works Association. At the time the investigation was made, there were no sewers in the town, and as all water was supplied through meters there was every inducement to curtail waste. There were 619 family supplies studied, the average family comprising five persons. This average family used 12,046 gallons during the year from the first or kitchen faucet, and this amount was used as a standard in rating the amount used by the remaining fixtures.

Additional faucets were estimated, as a result of the investigation, to add 20 per cent. each to the consumption, the first bath-tub 75 per cent., the second bath-tub 15 per cent.; the first water-closet 90 per cent., the second water-closet 40 per cent., set-tubs 20 per cent., and hose 90 per cent. The studies showed that faucets should be charged 75 per cent. of the family rate if used in groceries, 50 per cent. in dry goods stores, 75 per cent. in markets, dentists' offices and barber shops, 300 per cent. in fish stores, 250 per cent. in photographic studios, and the same charge for pharmacies as for dwellings. Water-closets are rated the same in business as in dwelling houses. The school rates for drinking water were placed equal to the domestic first faucet rate for each 200 pupils, and three times the family rate in the case of water closets. Church rates were found to be equitable when placed at half those for families. In power plants running ten hours daily it was found that a suitable charge would be 130 per cent. of the family faucet rate per horse-power of the boilers. Laundries were found to use per person about three times the first fixture con-

sumption of families. In greenhouses a fair charge seemed to be 85 per cent. of the faucet rate where the establishment was for forcing vegetables and the full faucet rate where it was run by florists. In the case of livery stables it was found desirable to charge 20 per cent. of the faucet rate for each horse, when carriages are not washed, 50 per cent. per horse where the carriages are washed without hose, and 75 per cent. if hose is used. In private stables, the full faucet rate was recommended for each horse. In trucking stables, one-third of the family rate seemed to be the proper charge for each horse, because so little washing of wagons is done.

Where families contained more than five members it was found that each additional person increased the consumption about 7 per cent.

In view of the fact that such a schedule is the best the writer has found as the result of correspondence and personal inquiry for a number of years, he considers its use in place of meters by any new works a deplorable step backward which will be deeply regretted.

KEEPING UP THE WORKS.

The problems which arise to perplex the superintendent of a water plant are so varied that but a few of the most common can be noticed here.

Where driven wells furnish the supply it is by no means unusual to have trouble with air when the pumps are running at their full capacity. If there is no air chamber on the suction pipe, with a pump to remove the air, the introduction of such an apparatus will prove probably beneficial. The present practice is to place one in the pumping station, as shown in Figure 31, but formerly they were often omitted. If the air chamber fails to remedy the pounding of the pumps, it is possible that some of the wells may be so shallow as to yield more air than water during periods of heavy draft, or the piping may have been injured. If each well has a valve by which it can be shut off, it will generally be possible to locate the trouble by shutting off one well after another until that which causes the trouble has been found. Slight leaks in the joints can be remedied by calking if they are lead or painting with thick asphalt paint if they are flange.

Anchor ice is one of the most annoying troubles of works drawing water from rivers and ponds. What it means can be best

shown by describing a couple of instances of its occurrence. The first case was reported by the late James B. Francis as taking place in a 1,750,000-gallon reservoir at St. John, N. B., in which the water had a maximum depth of 18 feet. At the time of the trouble there was about 8 feet of water and ice in the basin, and the surface was covered with ice except for about 250 square yards of open water directly over the inlet pipe. "On the evening of December 8th, 1882," Mr. Francis said, "the supply of water to this district suddenly ceased, and so continued for a short time until other connections were opened. On the following morning a hole was cut in the ice, which was about 6 inches thick, immediately over the outlet pipes, where a mass of ice was found, which is described as a kind of slush or minute particles of congealed water; on prying upon it, it floated up, and the bottom of it was exactly the shape of the strainer of the outlet pipe, showing that it rested upon it and completely closed up the outlet. The strainer is a copper rose, perforated with holes about ¼-inch in diameter, the marks of which appeared on the ice. The whole column when taken out, or as much of it as would hold together, was about the size of a barrel, and was composed of minute particles of ice, all standing on end, firmly adhering to each other." Mr. Francis concluded that the "ice which closed the strainer formed on the open water over the inlet pipe, was carried under the ice by eddies and currents, and continued in motion until it reached the strainers." The essential conditions for the formation of anchor ice, are, he held, that the temperature of the water be at the freezing point, and that of the air below that point; the surface of the water must be exposed to the air, and there must be a current in the water. The ice is formed in small needles on the surface, which are carried to the bottom by eddies. The little particles of ice are so small that when pressed against a stone or stick at the bottom, the tendency to adhere to the object on account of regelation is much greater than the exterior force tending to move the particle along. Other particles are caught in the same manner until the mass is produced which is called anchor ice. The adherence of the ice to the bed of the stream is believed to take place downstream from the place where it is formed, several miles, in fact, in the case of large rivers.

The Water Department of Ottawa, Canada, was troubled with this ice for many years. The pumps are driven by turbines which

became so clogged as to be stopped completely at times. One of the most serious difficulties occurred in December, 1894, when the entire station was rendered unserviceable for 6½ hours. It was shortly after midnight when the men on duty first became aware of the presence of the ice. The wheels began to revolve with less speed, but the slowing down had hardly been noticed before they all came to a sudden stop. Every man was put to work who could be made useful, and by 7 o'clock the first wheel began to move slowly. An hour later a second wheel was started, and at 9 o'clock a third commenced turning and after half an hour of jerky motion settled down to steady work at about half speed. The explanation of the trouble given by Mr. Robert Surtees, then city engineer, was that under certain atmospheric conditions, not particularly governed by the degree of cold, and when a wind is blowing, the little wavelets thus formed are frozen. The spicules of ice are carried down and pass through the gratings into the wheels, where they lodge. He noticed this condition never existed during the daytime, and apparently ceased with the slight change in the atmosphere caused by the rising sun. Attempts were made without success to keep the ice from the wheels by placing evergreen brush in front of the screen protecting them from sticks and other obstructions.

The most successful method of dealing with slush ice with which the writer is familiar was originated by Mr. John F. Ward while he was in charge of the water-works of Jersey City, N. J. So much ice of this sort collected at the screens of a reservoir that on one or two occasions the screens were broken. He made a small raft of 12x12-inch timbers which happened to be handy and moored it in front of the screens. This prevented the trouble entirely. His explanation of its success is that in consequence of the length of the line around the edge of the raft being, say, four times the width of the screen, the force available to draw in the ice is reduced in the same proportion. The ice collects in a mass around the raft and its cohesion is so great that the current is not strong enough to suck it under the timbers to the screen.

The method of dealing with anchor ice at submerged intakes was described in Chapter XV.

Hydrants are also a source of anxiety during the winter on account of their liability to freeze. The best way of preventing the trouble is to use long ones, so that the connection between them

and the street main will be well below the frost line. They usually have a drip hole so they can be drained into the nearest sewer, but if there are no sewers and the bottom of the hydrant is surrounded by any other material than very porous sand or gravel it will probably prove advisable to plug the hole. If this is not done the water will collect around the bottom of the hydrant and may freeze it. Such plugged hydrants must, of course, be kept free from water by a hand pump, which should be used immediately after every fire and at any other time an inspection shows it to be necessary. No matter how well the hydrants are drained they should be watched carefully during freezing weather, and thawed out whenever necessary by means of a portable boiler furnished with a rubber hose by which a jet of steam can be turned into the barrels.

At Bay City, Mich., the hydrants in the outlying parts of the city occasionally gave considerable trouble by freezing, where there was no adequate sewerage, and the ground water rose to the frost level. Mr. E. L. Dunbar found the most effective method of dealing with such hydrants was to bank them in autumn as high as possible, leaving room for the firemen to couple to the nozzles. They were frequently examined through the winter, and if ice began to form on them they were warmed thoroughly with steam or hot water, which usually protected them for several days.

In very cold climates particular precautions have to be taken with the tops of the hydrants. At Fredericton, N. B., Mr. Alexander Burchill reported in 1892 that the principal trouble was with the screw and stuffing box. He avoided it by taking off the caps of the hydrants every fall and running a clamp, made specially for the purpose, through the opening of the hydrant. It was secured there and held the valve rod firmly in place. The stuffing box, female screw and other parts were then taken out, wiped dry and clean, covered with kerosene and returned to their places, after which the clamp was taken out. The stuffing box packing was kept soft all winter by occasionally saturating it with kerosene, and no hydrant was opened during the winter if it could be avoided. As soon as water was turned off after a fire, the hydrant was at once attended to, and the stuffing box and other parts taken out, dried and oiled. At Woodstock, in the same province, Mr. Donald Monro started the tops of hydrants which stuck by means of circular pieces of canvas, 12 inches in diameter and soaked in

paraffine. One of these was placed on top of a hydrant, there being a hole to allow the nut to come through, and then set on fire. The heat always started the parts.

Frozen services are generally thawed out by means of small pipes inserted in them from within the house. Steam is turned into the pipes and the ice melted out in this manner. It is generally necessary to start the ice only, as the pressure of the water will force most of it out. In some places, as in Providence, the water department has a portable steam boiler from which run a number of small rubber pipes. A long 1-inch iron pipe is attached to each hose and furnished with a handle near the point of connection. One of the iron pipes is held vertically over the service pipe to be thawed out, near its junction with the street main. Steam is then turned on and the ground is quickly thawed, so that the pipe can be forced down by the handle like an auger to the service pipe. The steam then thaws out the pipe, and, as two or three lines of hose can be operated at once, it generally requires but 15 to 20 minutes to free a pipe. The most unique method of thawing services with which the writer is acquainted was described as follows by Mr. C. K. Walker, of Manchester, N. H.: "We have tried various ways, and now we dig a hole 2 feet deep and about 3 feet across; we put in some lime and pour on some water, go home and go to bed, and next morning everything is all thawed out."

The method of thawing pipes by electricity, which was introduced at Madison, Wis., in the winter of 1898-99 by Messrs. Jackson and Wood of the University of Wisconsin, is one of the most important improvements in water-works practice in recent years. The directions for using it, issued by the university, are as follows:

The current which is required for satisfactorily thawing service pipes up to 1½ inches in diameter is from 200 to 300 amperes. The source of current should have a pressure of not less than 50 volts. Where electric light lines carrying alternating currents are available, a transformer or transformers in parallel may be used as a source of current. It is very important that direct connection of pipes to house lines be avoided, on account of the danger of fire in which the house is placed by such a connection. Where alternating currents are not available continuous current feeder lines may be used, but these should be entirely separated from the distributing network of conductors.

Where an alternating circuit is used a fuse box should be placed

in each wire leading from the feeders to the transformer and an ammeter connected in one of these lines. The secondary leads or wires from the transformer should be No. 3 B. & S. gauge or larger. In making connection to the pipes, one of the secondary leads should be taken into the house to which the frozen service runs, and contact made by some form of metal clamp or simply winding the conductor tightly about a faucet or exposed water pipe. The other secondary lead should be put in contact with the water system outside of the house in a similar manner. This contact may be made at a hydrant, an adjoining service box, or a pipe in a neighboring house. Where there are two houses near together, each with frozen service pipes, the two secondary leads may be connected to the pipes within these houses and both frozen service pipes thawed out at once.

While the thawing process is going on, the faucet should be open in the house to which the service pipe runs. In one of the secondary leads should be inserted a water resistance which may consist of a bucket of water containing a bowlful of salt and two sheet-iron or copper plates, to which the ends of the severed lead are attached. This serves to control the current. When all connections are made, the plates are placed in the bucket and are then moved towards each other until the ammeter records a proper current. If the primary pressure is 1,000 volts and the secondary pressure 50 volts, the current should ordinarily approach 15 amperes. If the primary pressure is 2,000 volts and the secondary pressure 50 volts, the ammeter reading should ordinarily approach $7\frac{1}{2}$ volts.

Water does not ordinarily begin to flow in less than 10 minutes, but it may be nearly an hour before it starts. The frozen pipes are often split by the ice and begin to leak as soon as they are thawed, so it is desirable to have a plumber where he can be readily called in case such a leak is discovered.

The water pressure at the hydrants of some plants is tested every spring and fall by means of a pressure gauge previously examined with care to insure its accuracy. Such an inspection not only furnishes a good indication of the number of fire streams which may be reasonably expected, but also reveals any leaks in mains. For example, an inspection of this sort at St. John, N. B., showed a fall of 10 pounds at one hydrant below the reading taken six months before. A search was made and a leak was finally found

in a broken 4-inch pipe under an old wharf. This was repaired, but the gauge still showed a diminished pressure. Another search was made, which revealed a second broken 4-inch pipe; when this was repaired, the pressure returned to its normal amount.

In towns where there is an electric railway with tracks in the same streets as the water pipes, the latter are in danger of damage by electrolysis, which pits and eats through the metal as shown in Figure 53. This engraving was made from a photograph of a pipe destroyed by this action at Zanesville, Ohio.

There is nothing whatever mysterious about electrolysis. It is one of the elementary laws of electricity that when an electric current can pass by two circuits or routes to a given point, the current will be split into two parts in reversed proportion to the resistance of the routes; the greater part will pass by the route of least resistance and the smaller part by the other. In the case of

FIGURE 53.—THE RESULT OF ELECTROLYSIS.

the ordinary overhead trolley road, the electricity is sent from the power station through the overhead wires to the trolley of the car. It passes from the trolley through the wiring and motors of the car to the rails, and then nominally through the rails back to the power house. In order that the rails may return the current satisfactorily, their adjoining ends are bonded by copper wires and are very often connected to copper wires laid in the roadbed and running to the switchboard of the power station. The earth, particularly where it is moist, is a fair electrical conductor, and the water and gas pipes buried in it also offer little resistance to currents. Hence it frequently happens that a considerable portion of the current leaves the rails, passes through the earth and travels along a buried pipe until it reaches a point where the rails again offer a route of smaller resistance. At this place some of the current will leave the pipes and go back to the rails.

Where an electric current leaves a metallic conductor and passes through earth or a liquid to another metallic conductor, the metal of the first conductor is eaten away, a phenomenon utilized to electroplate various objects; where an electric current leaves a water pipe the metal is eaten away in just this manner. It is obvious that the remedy is to keep the current from the pipes as far as possible, or where this is impracticable to return it from the pipes by heavy metallic conductors which will prevent the electrolytic action, for this does not take place except where the metallic circuit is interrupted.

The railway companies should be just as much interested as the water departments in keeping the return currents on the tracks, for the wandering electricity means waste of power and consequent loss of income. Well-bonded rails and large conductors from the rails to the power house will accomplish a great deal in preventing electrolysis and saving current. These precautions will not do everything, however, and where the hydrants are found to be strongly positive to the rails they should be bonded with the latter by heavy copper wires, or, in the case of important pipes, the latter should be bonded to heavy copper conductors leading directly to the switchboard in the power station. The trouble usually begins with the service pipes and rarely occurs more than a quarter of a mile from the power station, and these precautions, although not absolute preventatives, are fairly good remedies. If the trouble is marked, the advice of an engineer should be obtained at once before the water plant is seriously injured. The fee for the advice is but a trifle compared with the loss due to the destruction of the street mains.

Setting meters should always be done by the water department, as previously mentioned. There are many methods of placing them, which were well summarized in 1896 in a report of a committee of the American Water-Works Association which was substantially as follows: In all cases a stop and waste cock should be placed between the meter and street main and a stop cock between the meter and the house pipe. This is of great advantage in removing meters for testing or repairs. When meters are set in cellars, they should be placed as near a wall as possible, to lessen the liability of connections being made ahead of the meter, and they should be accessible for reading or removal. Where there is danger from frost, the meter should be encased in a wooden box and

packed with sawdust, mineral wool or other non-conducting material. When it is placed in sidewalks or yards, it should be set in a brick pit or an iron or wooden box, large enough to allow its removal without disturbing the box. The pit or box should have an iron cover with a locking device. The meter should be set deep enough to avoid freezing, and, if necessary, packed with sawdust or other suitable material. Extension dials, which bring the counters to within a few inches of the surface, are a convenience in taking the readings.

Cleaning mains is usually accomplished by opening the blow-offs or hydrants and allowing the water to flush out the sediment. The work should be done on sections of the mains, one after the other, so that but a portion of the consumers are affected at one time. In case ordinary flushing will not answer, wooden balls are sometimes inserted in the pipe and forced along by the pressure of the water to the end of the line to be cleaned. Special arrangements must be made for inserting and removing the balls. Mr. Charles Hermany once used 5-inch wooden cubes for this purpose. Lead plugs of different weights were placed on opposite corners of each cube, so as to make it of approximately the same weight as an equal volume of water or a trifle heavier, the object being to have the block strike the bottom as well as the top of the pipe.

What are known as go-devils have also been employed in a few cities. According to Mr. Dexter Brackett those used in Boston consisted of a flexible central shaft about 3½ feet long, composed of coiled steel springs connecting small castings, to which were hinged two sets of steel scrapers arranged radially around the shaft about 12 inches apart. The scrapers were kept against the sides of the pipes by coiled springs, which allowed them to turn back to pass taps or other firm obstacles. Back of the scrapers were two rubber pistons, placed about 2 feet apart, so as to ensure a pressure on the machine when it was passing branches. A section was cut out of the pipe long enough to receive the scraper, which was then inserted and the joints made with lead in the ordinary manner, except that clamp sleeves were used, so that the section could be again easily removed and the scraper inserted if desired. A simliar piece was cut from the pipe at the lower end of the main to be cleaned, and the scraper was forced through the pipe by the ordinary water pressure, which varied from 30 to 45 pounds.

As occupants of buildings on the line of the pipes were without water while the work of cleaning was in progress and it was not thought advisable to pass the scrapers through valves, the pipes were cleaned in lengths averaging 1,000 feet. The scraper generally passed through this distance in from three to four minutes. In a few instances it was stopped by obstructions in the pipe, the one causing the most trouble being lead which had run into the pipe at a joint. The water issuing from the open end of the pipe was the color of ink for five to ten minutes after the scraper had passed through, and it was allowed to run to waste until it became clear; after this the section of the pipe was replaced and the valves opened. Some difficulty was experienced from the stopping of service pipes and house plumbing by rust forced into the pipes by the pressure of the water following the scraper, but this difficulty could generally be overcome by applying a force pump to the house plumbing and forcing the obstructions back into the main. The scraping doubled the discharging capacity of the pipes.

By this method the tubercles were removed from 58,000 feet of 6-inch pipe at a cost of 14 cents per foot, and from 20,300 feet of 12-inch pipe at a cost of 20.6 cents per foot. These prices include 5 cents per foot royalty for the right to use the scraper.

The approximate capacity of service pipes is shown by the accompanying table, which is taken from data compiled by the Boston Water Department.

Approximate Discharge in Gallons per Minute of Service Pipes of Different Diameters and Lengths. H=Head in Feet; L=Length of Pipe in Feet.

H equal	Diameter of Pipe in Inches.								
to	½	⅝	¾	1	1¼	1½	2	2½	3
10L	20	34	54	112	195	308	632	1,100	1,745
8L	18	30	49	100	175	275	566	988	1,560
6L	15	26	42	87	151	238	488	855	1,350
4L	12	22	34	71	123	195	400	700	1,100
2L	9	15	24	50	87	138	283	494	780
1.5L	7.5	13	21	43	76	119	250	428	675
L	6	11	17	35	62	97	200	350	555
L÷2	4.5	7.5	12	25	44	69	141	247	390
L÷4	3	5.5	8.5	18	31	49	100	175	275
L÷6	2.5	4.5	7	14.5	25	40	82	143	225
L÷8	2.2	4	6	12.5	22	34	71	123	195
L÷10	2.0	3.5	5.5	11	19	31	63	110	174
L÷20	1.2	2.2	3.5	7	13	21	45	80	130
L÷40	0.8	1.5	2.2	5	9	14	30	54	90

In closing this chapter, the writer wishes to emphasize the importance of keeping clear records in the office of the water de-

partment of every feature of the administration, financial and physical. There should be a complete set of plans of the works, books showing the size and *exact* location of every pipe, house service, hydrant and valve, so made out that they can be found immediately, even when the ground is covered by snow. Each gate should be located when possible by measurements to at least two permanent points.

In the pumping station, a book should be kept showing the time of each man, the number of hours each pump runs and the strokes or revolutions it makes, the amount of fuel used, the suction and pressure gauge readings at half-hour intervals, full data of all fire service, and the quantity of supplies received.

The financial books of the department have to be kept to conform with the ordinances under which the department is managed. There is no doubt that the best method of opening them is first to obtain specimen sheets from departments of the same size which have been organized for a number of years, and, second, to turn this material over to the best accountant in the community with a request to draw up a set of books fitting the local conditions. No general forms can be employed satisfactorily because of the variation in local conditions which renders them but partly useful.

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